CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

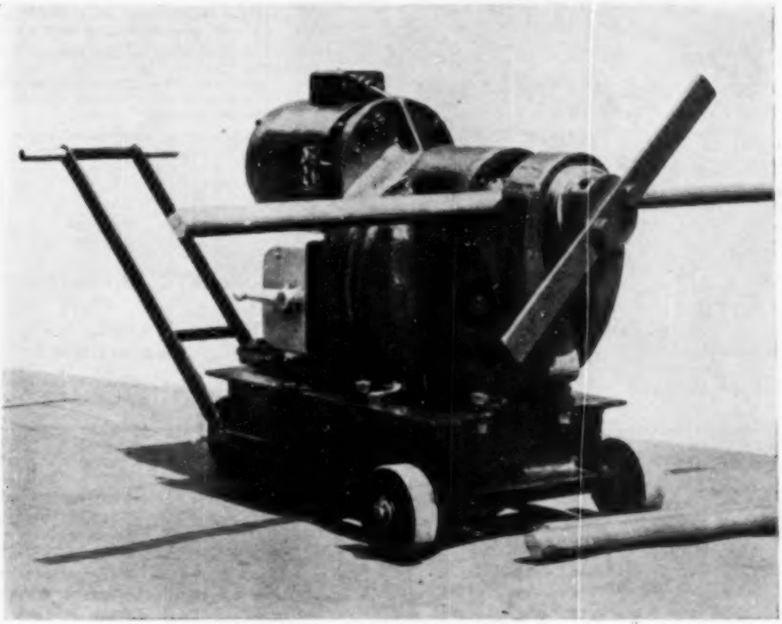
AUGUST, 1950. Vol. XLV, No. 8

FORTY-FIFTH YEAR OF PUBLICATION

PRICE 1s. 6d. ANNUAL SUBSCRIPTION 18s., POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 513 ISSUED MONTHLY *Registered for Canadian Magazine Post*

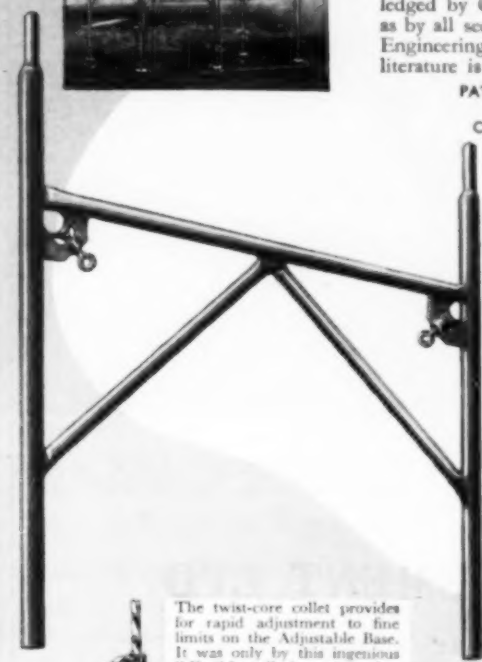


KWIKFORM

... the Unit-Frame Scaffolding System ...

. . . has revolutionised all previous conceptions of scaffolding. Economies obtained from its usage have been officially acknowledged by Government Authorities as well as by all sections of the Building and Civil Engineering Industries. Fully descriptive literature is available on application.

PATENTS GRANTED AND PENDING IN ALL PRINCIPAL COUNTRIES OF THE WORLD.



Illustrated above is the adaptability of our Unit-Frame System to extremely difficult site conditions.

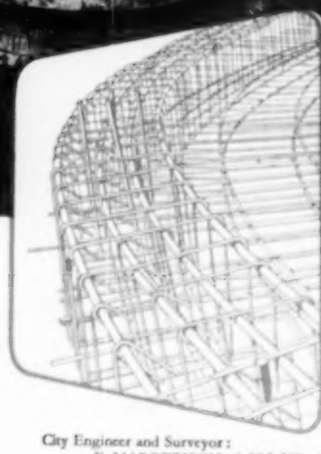
The twist-core collet provides for rapid adjustment to fine limits on the Adjustable Base. It was only by this ingenious "Kwikform" invention that the scaffolding problems of the building industry could be brought within the scope of full-scale factory prefabrication with all its benefits.

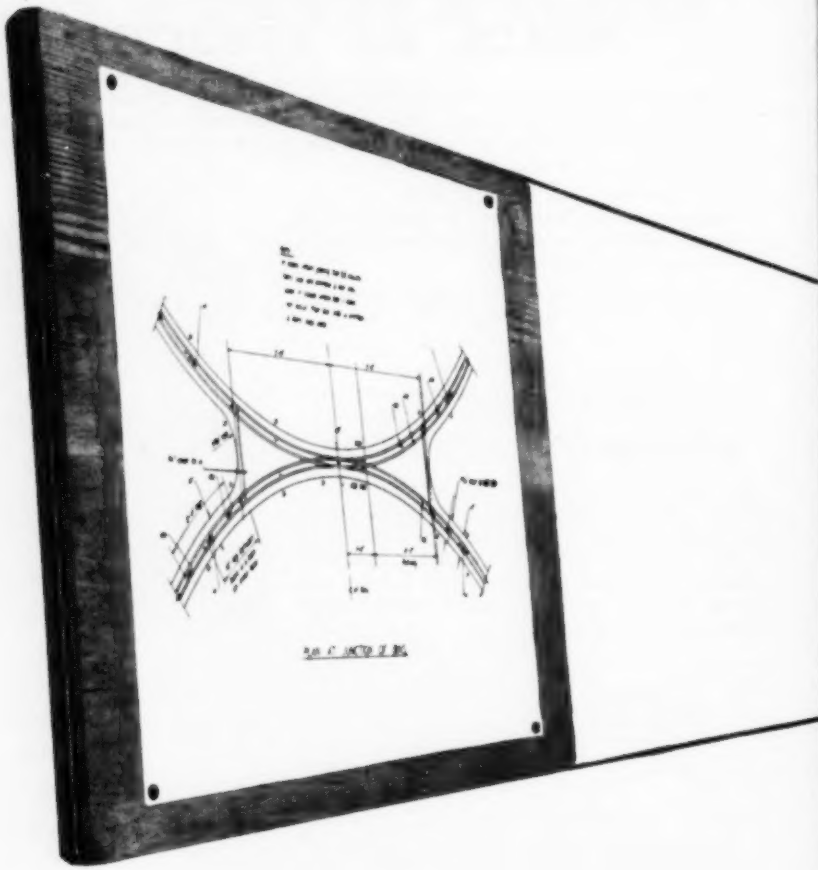
The above photograph of Nechells Power Station reproduced by permission of the British Electricity Authority shows the application of our Unit-Frame System to external and internal large-scale constructional operations.

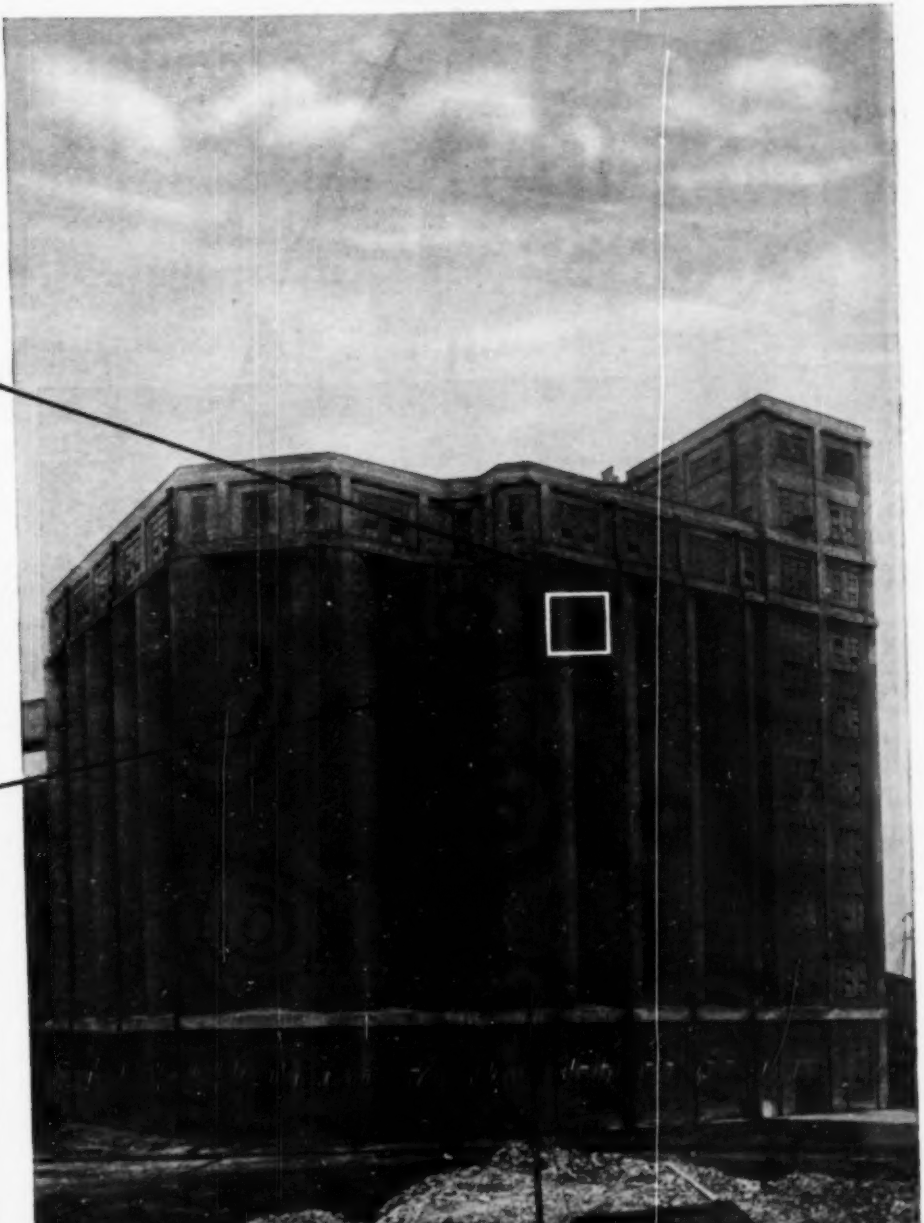
KWIKFORM LTD.

WATERLOO ROAD, BIRMINGHAM, 25. *London Office: 66 Victoria Street, S.W.1*

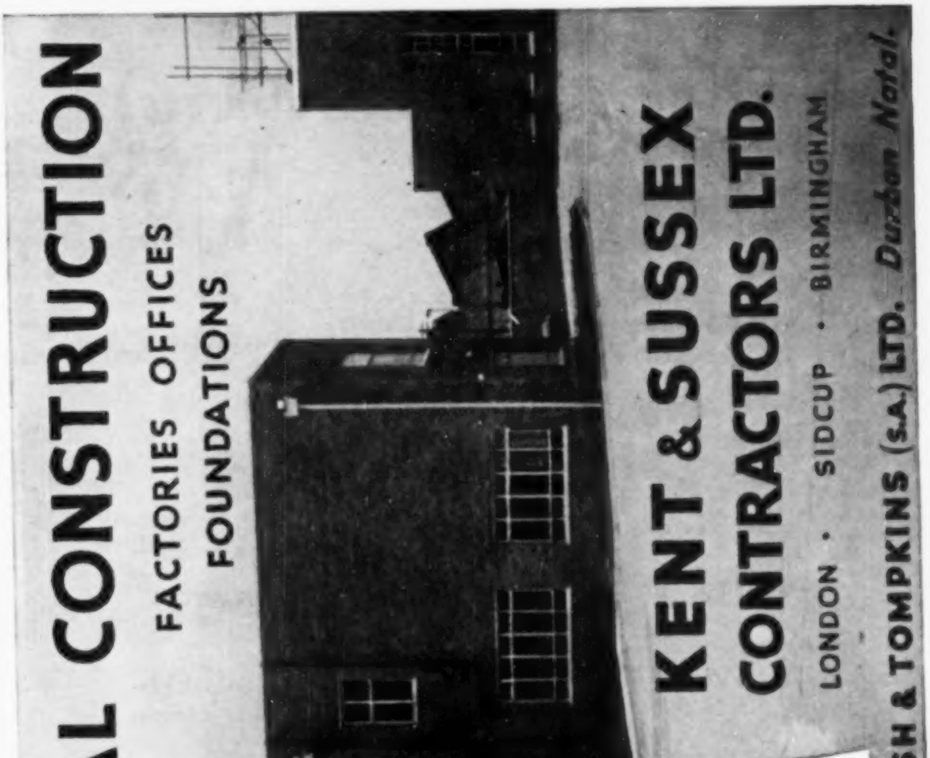
L.G.B.

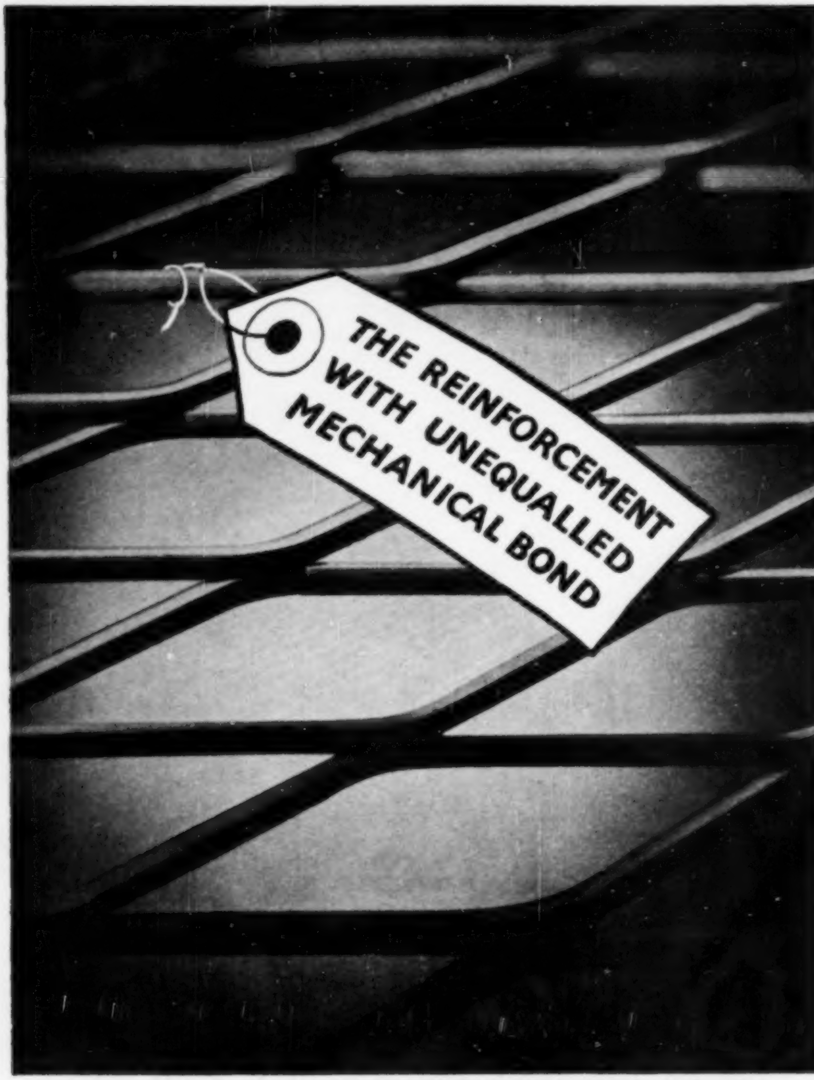






M-W.608


INDUSTRIAL CONSTRUCTION
FACTORIES OFFICES
FOUNDATIONS
KENT & SUSSEX
CONTRACTORS LTD.
LONDON · SIDCUP · BIRMINGHAM
Government Factory and Administrative buildings at Chadwell Heath, Essex, by Kent & Sussex Contractors
Associated Company - RUSH & TOMPKINS (S.A.) LTD. Durban Natal.


THE REINFORCEMENT
WITH UNEQUALLED
MECHANICAL BOND


We run our deliveries
to suit the progress
of your job
In the good old days when steel was plentiful we had to sell Isteg on its merits as a bar. These days its merits are taken for granted and the most important thing about Isteg is the service which stands behind it.
If you design in Isteg you can rely upon it being delivered when and as you need it, carefully bundled and accurately labelled for easy handling on the site. What's more you'll be helping to make more steel available for other urgent needs.
for Concrete Reinforcement
ISTEG
STEEL
ISTEG STEEL PRODUCTS LTD. (SALES)
43 UPPER GROSVENOR ST. LONDON W.1. Tel.: GROsvenor 1216
ISTEG is manufactured by Guest Keen & Nettlefolds (Cwmbran) Limited, Cwmbran. McCall & Co. (Sheffield) Ltd., Templeborough, Sheffield. The United Steel Companies Ltd., Sheffield and ISTEG STEEL PRODUCTS LTD.

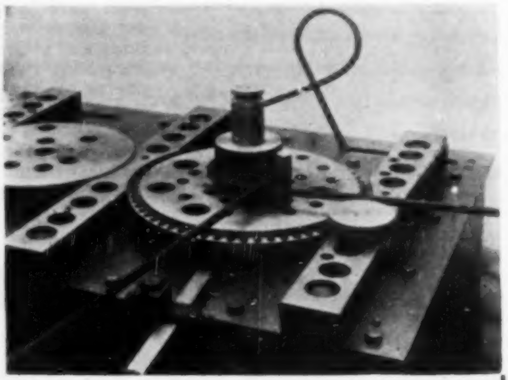
CONCRETE PROOFING CO. LTD.

GUNITE SPECIALISTS

Expert advice and schemes submitted for gunite work of every kind. Complete information on the various uses of gunite will be gladly sent on request.

100, Victoria Street, Westminster, S.W.1 Telephones: VICTORIA 7877 and 8278

ARD-50 BAR BENDER



This model is a double disc machine which will bend all diameter bars from ¼ in. to 2 in. It is supplied complete with standard accessories, and fitted with an electric motor, petrol engine, or a petrol-paraffin engine. The automatic control is governed by the fluted member (Brit. Pat. No. 469649) which permits the desired bending angle to be set mathematically. It is superior to other types in practical speed, accuracy, simplicity, and economy, and is invaluable for repetition bending. The illustration shows the bending of an angle loop in one operation. The accessories required for this are included in the standard accessories for all machines supplied. We also supply, at extra cost, a special device for bending hoops and spirals, and formers and back rests for special steel, such as "Isteg," "Square Grip," "Twisteel," etc. This model is in full production for **SALE or HIRE.** We also supply heavier and lighter power bending machines, details of which will be sent on request.

CEMENT & STEEL, LTD. SECOND AVENUE, CHATHAM, KENT

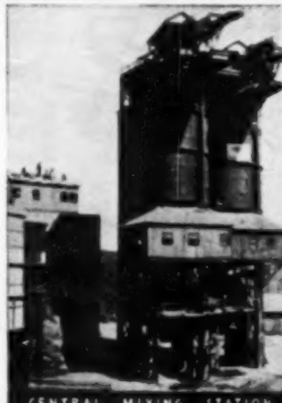
Telephone: Chatham 45580 *Telegrams and Cables: Cembelgi, Chatham*

BLAW KNOX IN WORLD BUILDING AND CONSTRUCTION

Castelo Do Bode Dam

This great engineering project on the River Zezere is part of the Portuguese Government's plan to provide hydro-electric power to feed a grid system for the electrification of railways from Lisbon to Oporto and for industrial development of the Country. The Central Mixing Station was supplied and erected by Blaw Knox and this contribution to a great undertaking is symbolic of the high quality construction equipment and integrity of service offered to engineers in the world-wide effort to build more, build faster and build to last.

BATCHING, MIXING AND BULK CEMENT PLANT · MOTOMIXERS · CONCRETE SPREADERS AND FINISHERS · CONCRETE MIXERS · ROADFORMS STEEL SHUTTERING · CONCRETE PUMPS · DEWATERING PLANT EXCAVATORS · EARTH MOVING EQUIPMENT



CENTRAL MIXING STATION AND BULK CEMENT STORAGE BINS

BLAW KNOX

BLAW KNOX LTD · 11 CLIFTON HOUSE · EUSTON ROAD · LONDON · ENGLAND

BK24

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XLV. No. 8. LONDON, AUGUST, 1950

EDITORIAL NOTES

The Trials of the Concrete Maker.

"THERE are periods in the growth of science when it is well to turn our attention from its imposing superstructure and to examine carefully its foundations." This sentence is from Professor Karl Pearson's "Grammar of Science" published in 1892. This advice is worth remembering in all branches of science, but it is seldom so opportune as when the word "scientific" is applied to the process of concrete making which, as at present practised, can have no claim whatever to be called scientific. It is a fundamental requirement of a scientific fact that it must be capable of exact repetition, or that nature repeats it exactly. It is a scientific fact that the earth rotates around the sun and so far as we know will continue to do so. It is a mathematical fact that two and two make four, and will continue to make exactly four so long as the words two and four have the same meanings as they have to-day. But whoever has made two or more batches of concrete that all have the same properties? At present this seems to be quite impossible, even in a laboratory, and it is generally necessary to assume that the average strength of three or more samples is the true strength of the material being tested. This lack of uniformity of the strength of concrete is expensive. It is, for example, the reason why 1 : 2 : 4 concrete can, according to the British Standard Code of Practice, be assumed to have a crushing strength of only 3000 lb. a square inch after it has matured for twenty-eight days, although it is quite possible to achieve a crushing strength of more than 10,000 lb. a square inch with the same materials, and in spite of the fact that most of the concrete will have a much higher crushing strength than 3000 lb. per square inch.

On the title-page of the first edition of Professor Pearson's book is printed the motto "La critique est la vie de la science". This is a truth that is brought home to the concrete maker in a paper read before the Reinforced Concrete Association, an abstract of which is given elsewhere in this number. The author points out that concrete in ordinary work having an average crushing strength of 5000 lb. per square inch would be likely to range in strength from 7000 lb. to 3000 lb. per square inch—so unscientific is concrete making even when employment is made of all the methods known to-day of obtaining uniformity and when the British Standard Code of Practice requires that the materials used for preliminary cube tests must be measured exactly to one part in one thousand. The results of the preliminary cube tests are so variable, in spite of the very

exact measurement of the ingredients and the attempt to secure uniformity in their manufacture, that three cubes have to be made and tested and the average assumed to be the truth. Such tests may indeed be likened to Pearson's scientific "imposing superstructure" on which is based the strength of concrete which may vary by more than a hundred per cent. however much care goes to its making. The reasons for the impossibility of making the concrete in a structure of uniform strength throughout lie in the nature of the materials and the processes used. It is shown that Portland cement may vary in strength by 50 per cent. or more, particularly if it is obtained from different makers, although the weakest sample will meet the requirements of the British Standard Specification for Portland Cement. Tests on the grading of aggregates, on which the strength of concrete largely depends because it affects the amount of water required to produce a given degree of workability, show enormous variations. For example in the case of various consignments of coarse aggregate delivered to one construction site the proportion which passed a ⅜-in. sieve varied from 4 per cent. to 70 per cent., and in the case of sands used in the same work the amount which passed a No. 25 B.S. sieve varied from 30 per cent. to 64 per cent.; these variations in grading would alone result in a difference of 20 per cent. in the strength of the concrete. The effect of moisture in increasing the volume of sand may vary the strength of concrete by 10 per cent., because bulking due to moisture means that, if the sand is damp and is measured by volume, up to 30 per cent. less sand may be used in the concrete compared with occasions when the sand is dry or saturated. It is estimated that by the common methods of measuring by volume the variation in the water-cement ratio may be as much as 15 per cent., with a consequent variation of about 30 per cent. in the crushing strength of the concrete. Examination of a large number of cores cut from concrete roads suggests that different degrees of compaction obtained in actual work may result in a reduction of up to 50 per cent. in the strength of the concrete compared with the strength obtained if the compaction were complete. In the making of test cubes it is estimated that variations in the proportioning, consolidating, and so on may lead to variations up to 30 per cent. in the cube-strength on the basis of which assumptions are made of the strength that will be or is being obtained in the work.

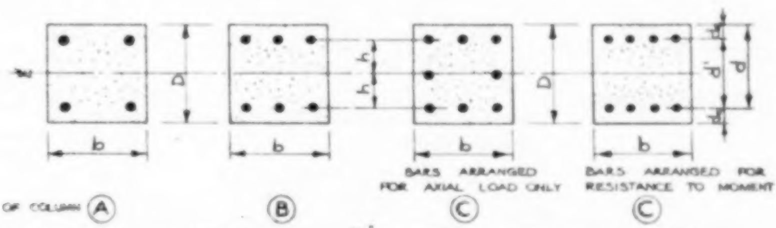
These causes of the variation of the strength of concrete are well-known, but it is well that attention should be drawn to them again and again. There can be few industrial procedures which give more variable results than those in common use in the production of concrete. It is fortunate that the extreme effects of all the factors that can reduce the strength of concrete do not in practice occur at the same time on the same structure. Much has been accomplished in recent years in devising methods by which greater uniformity of concrete will be produced. For example, the author of the paper referred to states that measurement of materials by weight will result in variations of not more than 4 per cent. in the water-cement ratio, and in the best conditions of measurement by volume the variation should not be more than 8 per cent. But obviously much remains to be done if concrete making is to be put on a scientific basis, and revolutionary ideas are wanted.

Square Columns Subjected to Combined Stress.

DESIGN IN ACCORDANCE WITH THE BRITISH STANDARD CODE.

By DONOVAN LEE, B.Sc., M.Inst.C.E., M.I.Mech.E., M.I.Struct.E.,
and OLIVER CUNDALL, B.Sc.(Tech.).

DESIGN charts based on permissible stresses other than those recommended in the British Standard Code of Practice (C.P. 114, 1948), "The Structural Use of Normal Reinforced Concrete in Buildings," may be of little value for use in designs to be made in accordance with this Code. The accompanying charts are applicable to the design of square (and rectangular) columns subjected to axial load only or to a direct load combined with bending. The columns con-



A_c = TOTAL AREA OF REINFORCEMENT   1½" CONCRETE COVER TO ALL BARS

COL Nº	TYPE	b × D	MAIN BARS	A_c	p
8A	A	8" × 8"	4 - ½" ϕ	0.785	1.226
8B	A	"	4 - ⅝" ϕ	1.228	1.92
8C	A	"	4 - ¾" ϕ	1.768	2.77
10A	A	10" × 10"	4 - ⅝" ϕ	1.228	1.228
10B	A	"	4 - ¾" ϕ	1.768	1.768
10C	A	"	4 - ⅞" ϕ	2.404	2.404
12A	A	12" × 12"	4 - ⅝" ϕ	1.228	0.8527
12B	A	"	4 - ¾" ϕ	1.768	1.23
12C	A	"	4 - ⅞" ϕ	2.404	1.669
12D	A	"	4 - 1" ϕ	3.142	2.18
14A	A	14" × 14"	4 - ¾" ϕ	1.768	0.903
14B	A	"	4 - 1" ϕ	3.142	1.61
14C	A	"	4 - 1⅛" ϕ	3.976	2.03
14D	B	"	6 - 1" ϕ	4.713	2.41

COL Nº	TYPE	b × D	MAIN BARS	A_c	p
16A	A	16" × 16"	4 - 1⅛" ϕ	3.976	1.553
16B	B	"	6 - 1" ϕ	4.713	1.841
16C	B	"	6 - 1⅛" ϕ	5.964	2.33
18A	B	18" × 18"	6 - 1" ϕ	4.713	1.455
18B	B	"	6 - 1⅛" ϕ	5.964	1.841
18C	B	"	6 - 1¼" ϕ	7.362	2.273
20A	B	20" × 20"	6 - 1" ϕ	4.713	1.178
20B	B	"	6 - 1⅛" ϕ	5.964	1.481
20C	B	"	6 - 1¼" ϕ	7.362	1.840
20D	C	"	8 - 1¼" ϕ	9.82	2.45
24A	B	24" × 24"	6 - 1⅛" ϕ	5.964	1.035
24B	B	"	6 - 1¼" ϕ	7.362	1.28
24C	B	"	6 - 1½" ϕ	10.60	1.84
24D	C	"	8 - 1½" ϕ	14.136	2.455

PERCENTAGE OF REINFT: $p = \dfrac{100 A_c}{bD}$

Fig. 1.

sidered are from 8 in. square to 24 in. square and are assumed to be of 1 : 2 : 4 concrete (not vibrated) for which the permissible compressive stress is 760 lb. per square inch under axial load only and 1000 lb. per square inch under direct load and bending. Three or four columns of each size are considered, the differences being in the amount of plain round mild steel reinforcement (*Fig.* 1). Considerations of economy generally favour small proportions of reinforcement, and there is therefore only a limited range of sizes and numbers of bars for each size of column. In the charts the amount of reinforcement is not less than 0·8

per cent. and the maximum about 3 per cent., although the Code permits up to 8 per cent.

Some methods of designing columns subjected simultaneously to direct load and bending assume the distance from the face of the column to the centre of the bars to be a definite proportion of the dimensions of the column, an assumption

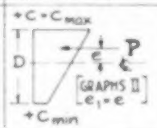
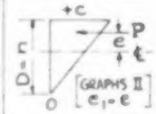
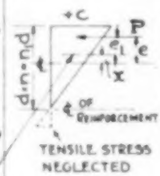
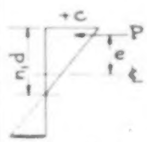
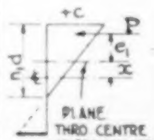
POINT ON GRAPHS	STRESSES (LB. PER SQUARE INCH)	FORMULÆ $m = 15$ BENDING TAKEN ABOUT AXIS OF GREATER MODULUS	STRESS DISTRIBUTION
POINT A	$C = 760$ $C_1 = 18{,}000$	$P = 760(bD - A_c) + 18{,}000A_c$ (lb) $M = 0$	AXIAL LOAD. UNIFORM COMPVE STRESS C
SECTION 1	$C = 1000$	$p = \frac{100A_c}{bD}$ $P = \dfrac{1000bD}{\frac{1}{1+0.14p} + \frac{6De}{D^2+0.42p(d')^2}}$ (lb)	
POINT B	$C = 1000$	$P = 7000A_c + 500bD$ (lb) $M = Pe$ [or M_1] $= 83.3bD^2 + \frac{3500A_c(d')^2}{D}$ (in-lb) (GRAPHS 1) [(GRAPHS II)]	
POINT C	$C = 1000$	$n_1 = 1.0$ $\frac{e}{d} = \dfrac{\frac{D}{2} - \frac{d}{3} + \frac{0.14p_1hd'}{d}}{d + 0.14p_1d'}$ $p_1 = \frac{100A_c}{bd}$ $P = 1000bd\left[\frac{1}{2} + 0.07p_1\left(1 - \frac{d_c}{d}\right)\right]$ lb GRAPHS 1: $M = Pe$ in-lb GRAPHS II: $e_1 = \frac{M_1}{P}$ in PLANE THROUGH CENTRE OF RESISTANCE $x = \dfrac{\left(\frac{p}{100}+1\right)d_c - \frac{pD}{200}}{2 + 0.29p_1}$	
SECTION 2	$C = 1000$	$\frac{e}{d} = \dfrac{-n_1^3 + \frac{3n_1^2D}{2d} + 0.9p_1\left(\frac{h}{d}\right)^2 - 0.03p_1\frac{h}{d}\left(n_1 - \frac{d_c}{d}\right)}{3n_1^2 + 0.9p_1n_1 - 0.45p_1\frac{D}{d} - 0.03p_1\left(n_1 - \frac{d_c}{d}\right)}$ $P = 1000bd\left[\frac{n_1}{2} + \frac{0.07}{n_1}p_1\left(n_1 - \frac{d_c}{d}\right) - \frac{0.075}{n_1}p_1(1-n_1)\right]$ lb GRAPHS 1: $M = Pe$ in-lb GRAPHS II: $e_1 = \frac{M_1}{P}$ in. $x = \dfrac{\frac{D}{2}\left(n_1 - \frac{p_1}{200}\right) - \frac{n_1^2d}{2} + \frac{p_1d_c}{200}}{n_1 + 0.145p_1}$	
POINT D	$C = 1000$ $C_1 = 18{,}000$	AS SECTION 2 WITH $n_1 = 0.454$	
POINT E	$C_1 = 18{,}000$	$P = 0$ $n_1 = \sqrt{0.021p_1^2 + p_1\left(0.15 + 0.14\frac{d_c}{d}\right)} - 0.145p_1$ $M = 9000A_c(d-y)$ in-lb $y = \dfrac{\frac{n_1^3d}{3} + 0.14p_1d_c\left(n_1 - \frac{d_c}{d}\right)}{n_1^2 + 0.14p_1\left(n_1 - \frac{d_c}{d}\right)}$ in	

Fig. 2.—Formulæ.

which is only correct by coincidence. If, as in the charts, the true dimension with relation to the size of the bar and 1½ in. of cover of concrete is used in the calculations a more accurate result is obtained at the expense of other limitations, such as the maximum stress and quality of concrete. The charts, however, enable a square column to suit most ordinary requirements to be selected without calculation when the applied load and moment are known. To conform to the Code the modular ratio is 15 and the formulæ are derived from the basic assumptions given in the Code. The stress in the reinforcement is 18,000 lb. per square inch for columns subjected to axial load (no calculable moment) and does not exceed 18,000 lb. per square inch under combined stress. The charts apply to

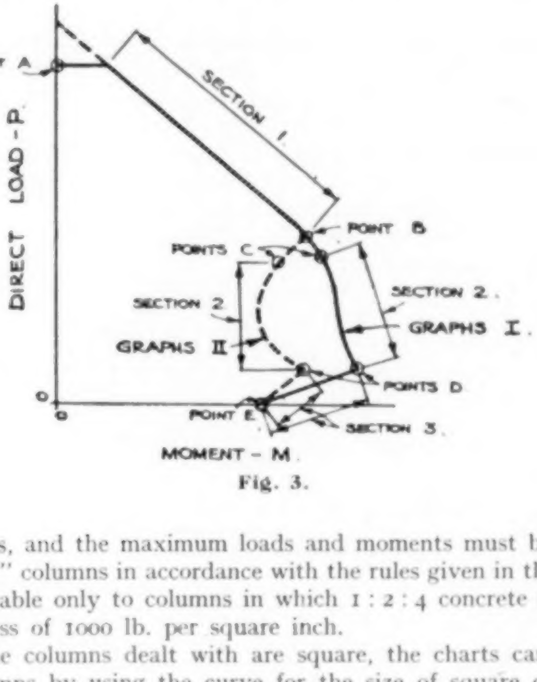
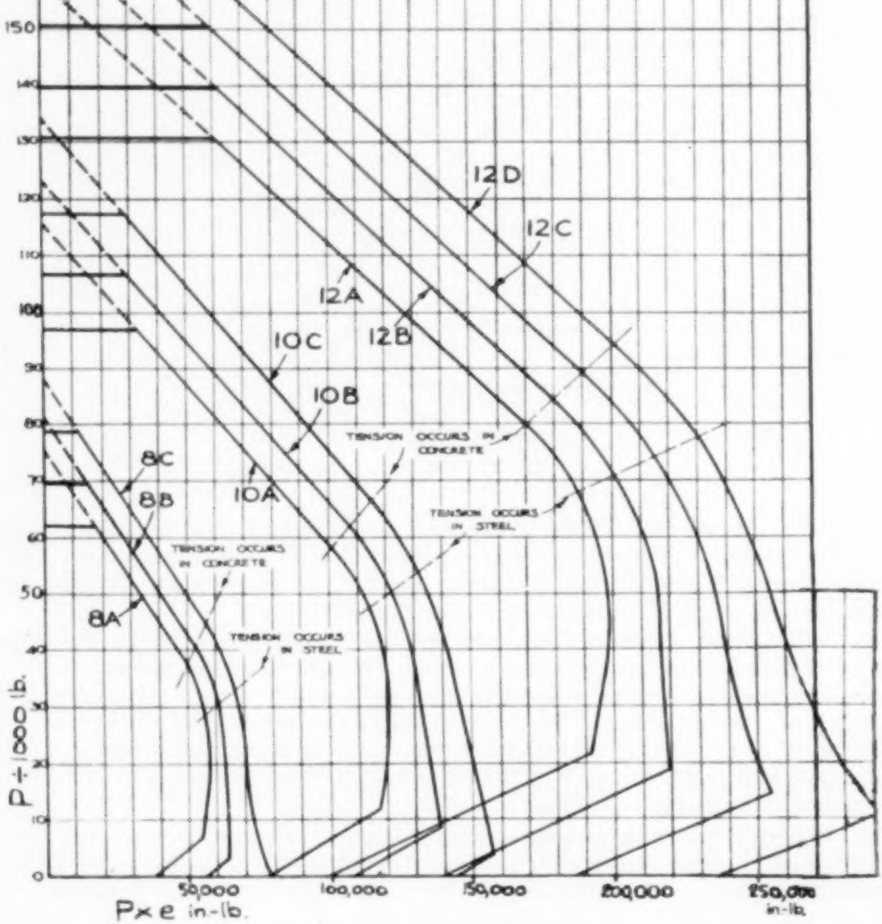


Fig. 3.

"short" columns, and the maximum loads and moments must be reduced for slender or "long" columns in accordance with the rules given in the Code. The charts are applicable only to columns in which 1 : 2 : 4 concrete is used or for a maximum stress of 1000 lb. per square inch.

Although the columns dealt with are square, the charts can be used for rectangular columns by using the curve for the size of square column of the same dimension in the plane of bending as the rectangular column. The moments, loads, and amount of reinforcement are reduced (or increased) in proportion to the width of the rectangular column to the width of the square column. The percentage of reinforcement will be the same as that in the square column, although the amount will differ.

Two series of graphs are given. Graphs I apply to the case when the position of the line of action of the load, that is the eccentricity e from the geometrical centre, is known. Graphs II relate to cases where the moment M and load P are known. The eccentricity e in this case is $\frac{M}{P}$ and is measured from the centre of resistance of the column. For small ratios of M to P when there is no tensile stress in the column, the centre of resistance and the geometrical centre coincide,



Graph 1 A.—Columns 8 in. to 12 in. square: Eccentricity known.

and for this condition Graphs I and II are identical. When tensile stresses are present, the two centres are not coincident and, as the ratio of M to P increases, the difference between the curves in Graphs I and II increases, but for large ratios the difference decreases until the curves are again coincident at the point corresponding to bending only ($P = 0$).

Explanation of Graphs.

Each curve on the graphs applies to all conditions from axial load acting alone (no moment) to bending moment acting alone (no direct load). Various conditions affect the analysis of each section of the curves as explained in the following.

GRAPHS I.—DIRECT LOAD AND ECCENTRICITY KNOWN.—Referring to the typical curve (*Fig.* 3), the stress diagrams, and the formulæ in *Fig.* 2, the conditions at critical points are :

Point A.—The load P is the greatest axial load which the column can carry with a uniform intensity of stress c of 760 lb. per square inch in the concrete and 18,000 lb. per square inch in the reinforcement. This condition also applies when the moment $M (= Pe)$ is small as shown by the horizontal part of the curve.

Section 1.—The moment combined with the load produce only compressive stresses, the maximum stress in the concrete being 1000 lb. per square inch.

Point B.—There is no stress at one edge of the column ; at the opposite edge the stress is 1000 lb. per square inch.

Point C.—There is no stress in the reinforcement near one edge of the column ($n_1 = 1$) ; the tensile stress in the concrete between this edge and the reinforcement is neglected. At the opposite edge, c is 1000 lb. per square inch.

Section 2.—The neutral axis is above the reinforcement near one edge ; this reinforcement is in tension and the stress therein does not exceed 18,000 lb. per square inch. At the opposite edge, c is 1000 lb. per square inch.

Point D.—The stress in the tensile reinforcement is 18,000 lb. per square inch, and the maximum stress in the concrete is 1000 lb. per square inch ($n_1 = 0{\cdot}454$).

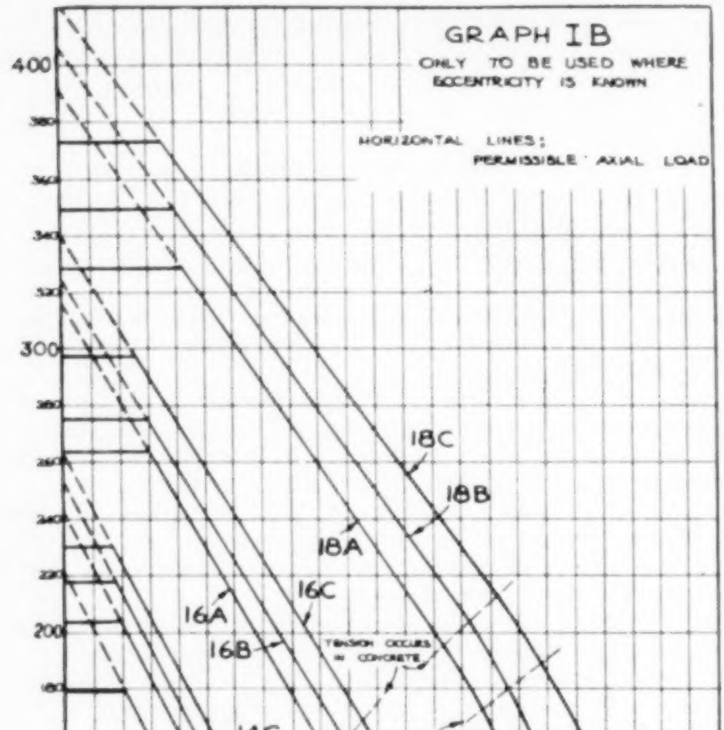
Section 3.—The stress in the tensile reinforcement is 18,000 lb. per square inch, but the maximum stress in the concrete decreases as the load P decreases.

Point E.—This is the limit of Section 3 when there is only pure bending on the column.

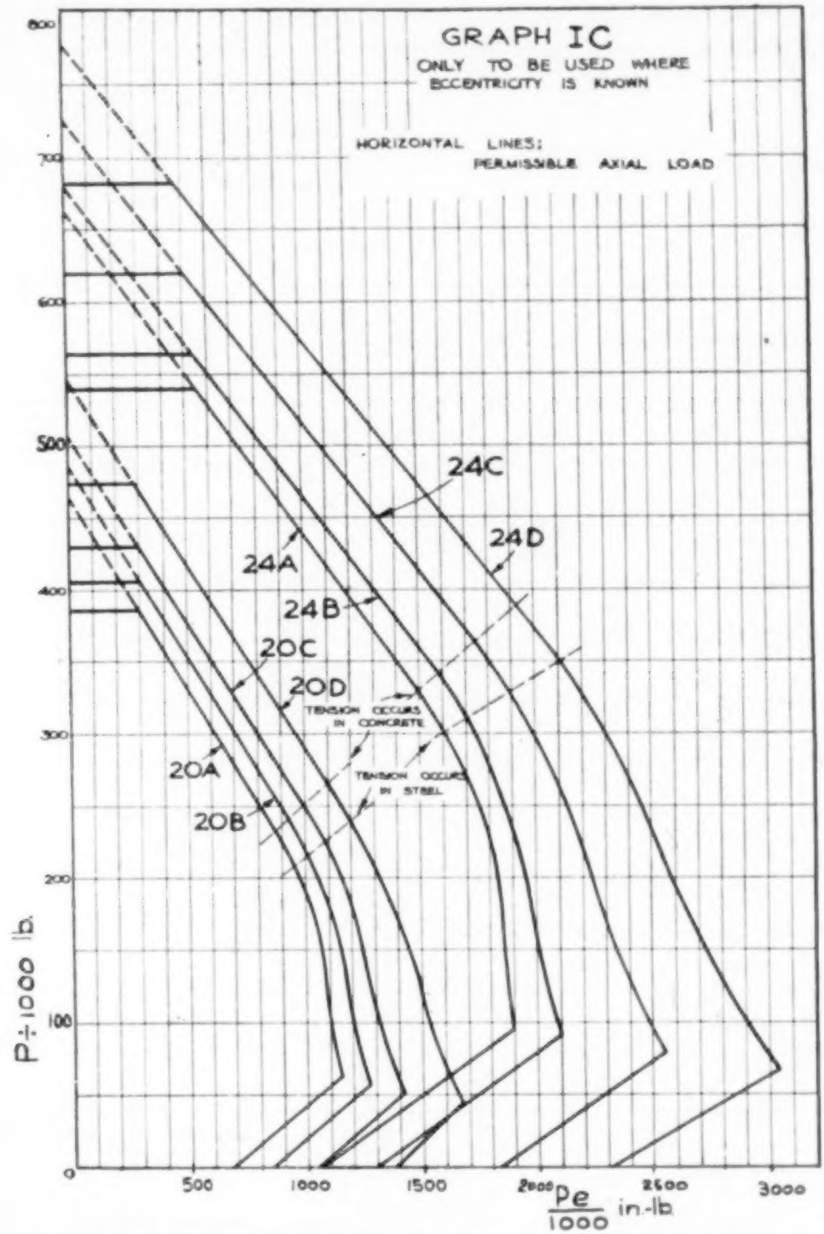
GRAPHS II.—MOMENT AND DIRECT LOAD KNOWN.—This is the case when the load acts at the centre of resistance of the column. The conditions at each of the points and sections are the same as for Graphs I, but the formula (*Fig.* 3) for Point C and Sections 2 and 3 differs because $e = e_1 + x$ where $e_1 = \frac{M}{P}$ and x is the distance between the geometrical centre and centre of resistance.

Application of Graphs.

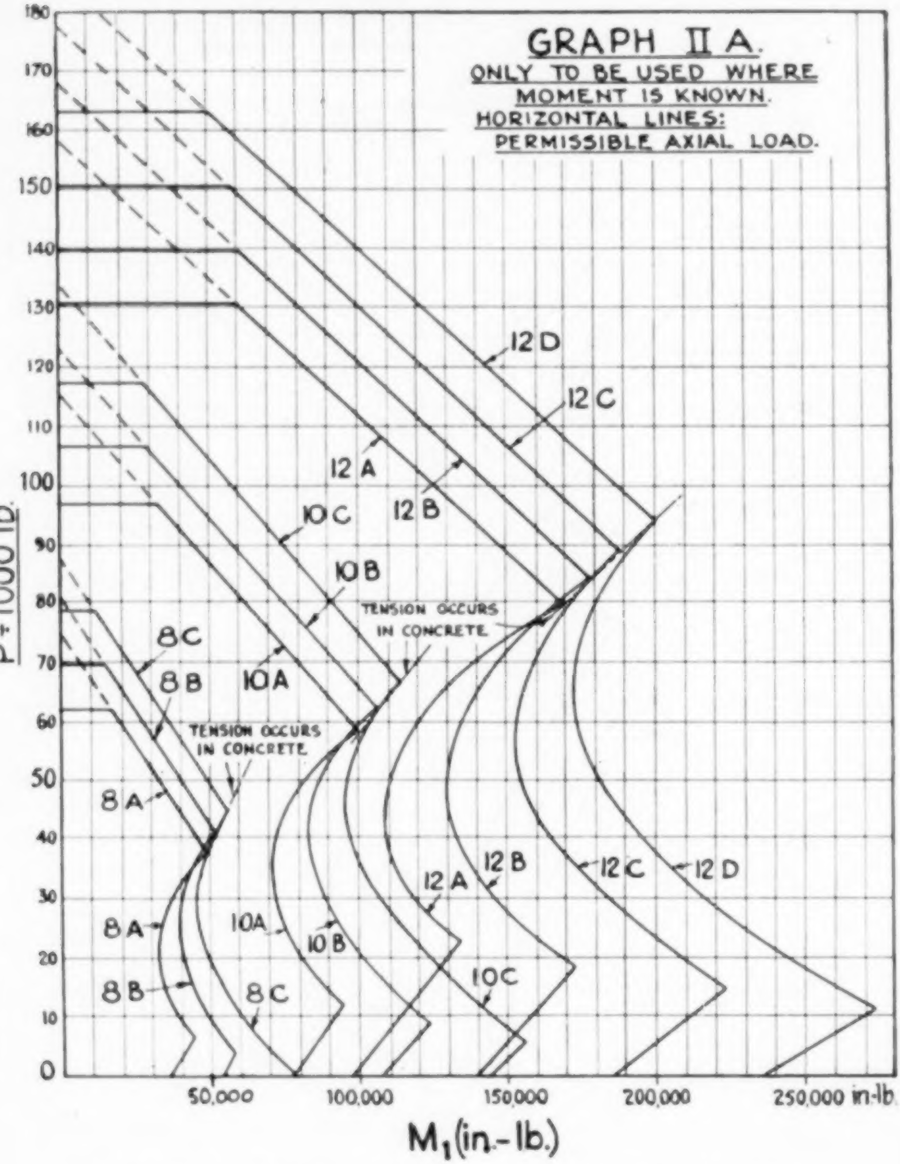
Graphs I apply to columns with connections on the sides for beams (such as brackets carrying precast beams, crane-rail beams, etc.), arches with eccentric thrusts, portal frames with permanent joints at the points of contraflexure, and other columns for which the line of action of the load is known.



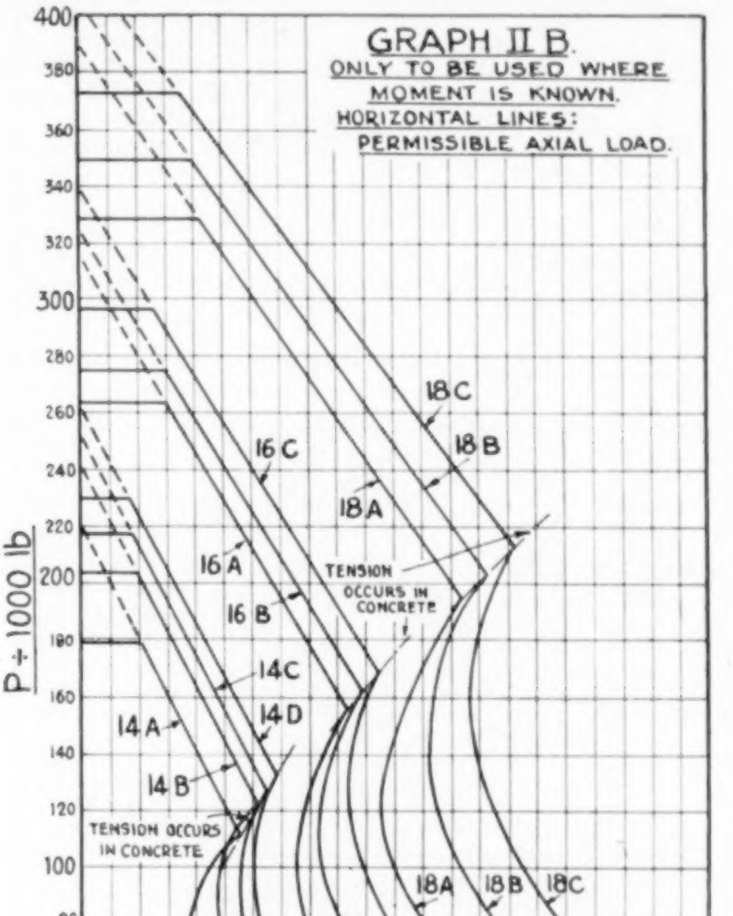
Graph 1B.—Columns 14 in. to 18 in. square: Eccentricity known.



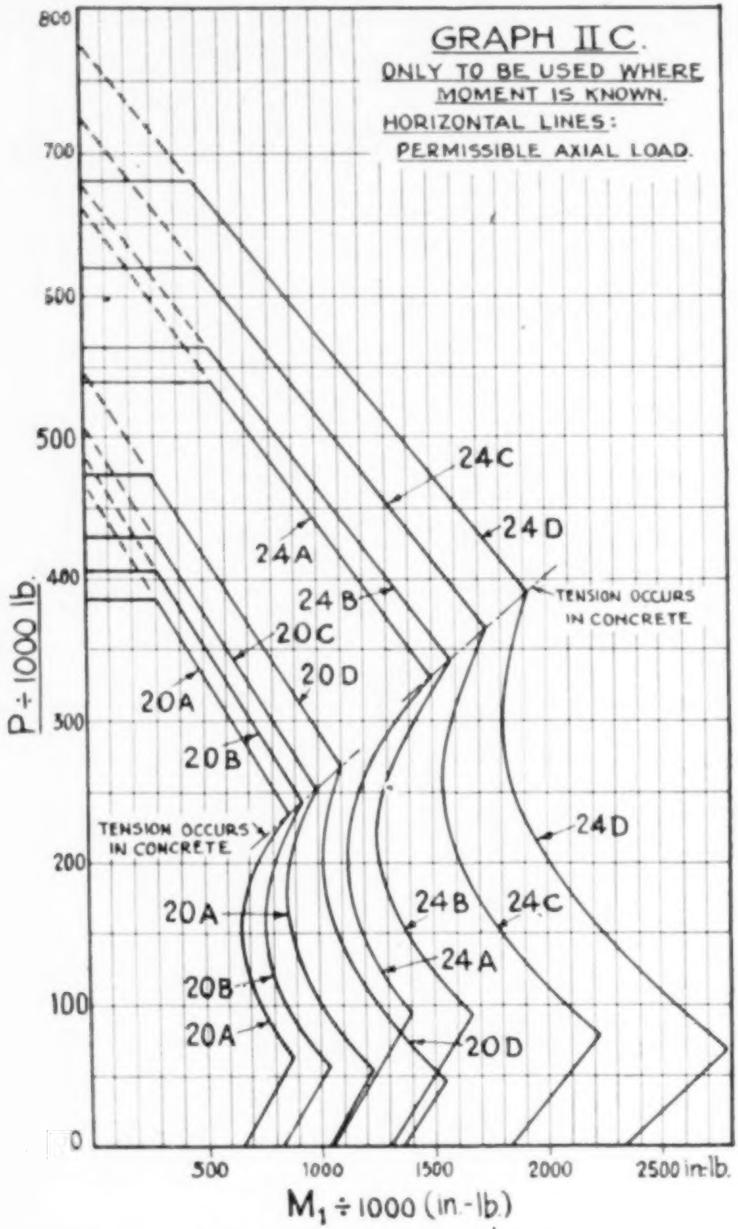
Graph I C.—Columns 20 in. to 24 in. square: Eccentricity known.



Graph II A.—Columns 8 in. to 12 in. square: Moment known.



Graph II B.—Columns 14 in. to 18 in. square: Moment known.



Graph II C.—Columns 20 in. to 24 in. square: Moment known.

Graphs II apply to the columns of frames and similar structures designed as indeterminate structures for which a moment and a load, acting at an indefinite point, are calculated.

The graphs can be used to select a column to resist a known load P acting at a known eccentricity e, or to resist a known moment M combined with a related load P. Thus if P is known to be 90,000 lb. and e is 2 in.,

$$M = 90{,}000 \times 2 = 180{,}000 \text{ in.-lb.}$$

and Graphs I apply. From Graph IA it is seen that a column 12 in. square is required. The reinforcement in column 12B is not quite enough, but that in 12C (four ⅞-in. bars according to *Fig.* 1) is sufficient.

If a column is subjected to a moment M of 600,000 in.-lb. and a load P of 80,000 lb., Graphs II apply, and, from Graph IIB, column 18A (18 in. square; six 1-in. bars) is satisfactory.

The graphs can also be used in the same manner as a check to ascertain whether a column is large enough for its purpose. For example, it is desired to know whether a column 12 in. square reinforced with four ⅝-in. bars (column 12A) can resist the combined effect of a moment of 60,000 in.-lb. and a load of 120,000 lb. Curve 12A in Graph IIA applies. Since the intersection of $M = 60{,}000$ and $P = 120{,}000$ is to the left of this curve, the column is satisfactory in accordance with the Code.

Concrete as a Protective Barrier for Gamma-Rays.

In the Editorial Notes in this journal for March, 1950, reference was made to research in the U.S.A. relating to the use of concrete as a protective barrier for radiation of gamma-rays. Further research on this subject is reported in the "Journal of Research" of the National Bureau of Standards of the U.S. Department of Commerce, and has special reference to gamma-rays from cobolt-60. It is stated that for effective protection of persons working near a source of radiation of this nature, the thickness of a concrete barrier should be 5 in. if the strength of the source is 10 millicuries (mc.), 13½ in. if 100 mc., 21½ if 1000 mc., and 24 in. if 2000 mc. The corresponding thicknesses of a lead barrier are 0·7 in., 2·4 in., 4 in. and 4·5 in. These thicknesses, which are based on broad-beam radiation, assume that the distance from the source to the barrier is 3 ft. 3 in., but less thicknesses are required if the distance is greater. For example, if the distance is 6 ft. 6 in. the corresponding thicknesses are 8¼ in. (1·4 in.), 16¼ in. (3 in.), and 19 in. (3·5 in.); the figures in brackets refer to lead barriers.

Coarsely-ground Cement for Experimental Road.

In order to get information on the relative durability of concrete roads in which coarse and fine Portland cements are used, the State Highway Commission of Kansas, U.S.A., is to use coarsely-ground cement for about 18 miles of a new 24-mile concrete road carrying considerable traffic including heavy vehicles. The coarsely-ground cement will be made as nearly as possible to match the Portland cement used in the district thirty years ago, while the remainder will be Type 1 finely-ground cement as made to-day. The experiment is being made as a result of an investigation made in the years 1940 to 1944, when an inspection of 1270 miles of concrete roads showed that those laid before the year 1930 were more satisfactory than those laid since; it was found that the older roads, although cracks had appeared, were sound and more durable. The specifications for the whole of the road are exactly the same except for the fineness of the cement.

Book Reviews.

"Strength and Elasticity of Materials and Theory of Structures." By W. H. Brooks. Volume I. (London: Macdonald & Co. (Publishers) Ltd. 1950. Price 12s. 6d.)

THE sole contents of this small book are described by the subtitle, "Solutions to Examination Questions of the University of London," since it contains fully-worked answers to 111 questions set in the subjects in the internal examination for the degree of Bachelor of Science (Engineering) from 1938 to 1948. In this respect the author is rendering a service to students, as the University no longer publishes model answers. The solutions, which are collected under different headings, are given with little, but useful, comment; one such is the statement that it is presumed from the wording of a particular question that what is required is the stiffness of a member in accordance with the old definition of ratio of deflection to length. None of the questions relates to reinforced concrete.

"Roads: Their Alignment, Layout, and Construction." By R. G. Batson. 1950. (London: Longmans, Green & Co., Ltd. Price 21s.)

IN his preface, the author states that this book is intended as a guide to the student whose studies include roads. Six chapters on the planning of roads are followed by a short chapter on foundations (mostly laboratory work), 12 pages on road construction, 16 pages on concrete roads, 24 pages on bituminous roads and pavings, 26 pages on private street works (mostly dealing with legal matters), and 8 pages on road bridges.

"Investigations on Building Fires." By T. W. Parker, P. W. Nurse, and G. E. Bessey. (London: H.M. Stationery Office. 1950. Price 9d.)

THIS booklet deals first with methods of estimating the greatest temperature that may have occurred during a fire. The methods are usually based on the effect of high temperatures on certain materials, but warning is given that an inaccurate result may be obtained if the material has been protected temporarily by another, or when the general temperature is estimated by the effect on a material a part of which may have been subjected to a very high local temperature. The second part deals with changes visible in concrete and cement mortar when subjected to high temperature. At certain temperatures—300, 600, 950, and 1200 deg. C.—the changes in colour of concrete and mortar with siliceous aggregates are sufficiently well defined to enable the temperature to which the material has been subjected to be ascertained. The change to red colour at 300 deg. C. is a useful guide.

"Einflüsse auf Beton und Stahlbeton." By K. Walz, H. Vierheller, and A. Kleinlogel. Fifth edition. (Berlin: Wilhelm Ernst & Sohn. 1950. 25 D.M.)

THIS book, in the German language, is in three parts in which, under more than 300 headings, are discussed the effect on concrete of various fluids and solids, and such influences as weather, internal temperature, methods of curing, etc. Special types of concrete, other materials, and methods of construction are described and several proprietary names are explained. The source and author of each entry is given.

"Stählerne Brücken." By G. Schaper. Vol. 1, part 1. Seventh Edition. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 18 D.M.)

ALMOST one half of this book (in the German language) describes very fully the properties, and in less detail the manufacture, of steel. Much of the remainder deals with riveted and welded joints as preliminaries to the consideration, presumably in subsequent volumes, of the design and construction of steel bridges. There is a short description of the types of such bridges and a brief history commencing with the first iron bridge in the world at Ironbridge, Shropshire.

Books Received.

"The Technical Aspect of the Design of Roads." By S. Champion. 1950. (London: Technical Press, Ltd. Price 30s.)

"Publications of the International Association for Bridge and Structural Engineering." Vol. 9. (Zurich: Verlag Leemann. 1950. No price stated.)

"Concrete Shell and Barrel Roofs." Second Edition. (Obtainable free from the Cement and Concrete Association, 52 Grosvenor Gardens, London, S.W.1.)

"Questions and Answers on Concrete Road Construction." (Issued by the Cement and Concrete Association for the Road Research Laboratory of the Department of Industrial and Scientific Research. Obtainable free from the Association, 52 Grosvenor Gardens, London, S.W.1.)

"Building Science." By D. A. G. Reid. Vol. 1. 157 pages. 5 in. by 7½ in. (London: Longmans, Green and Co. 1950. Price 6s.)

"Book-keeping and Accountancy for Private Companies." By Owen J. West. 1950. (London: Jordon & Sons, Ltd., Price 10s. 6d.)

"Bonusing for Builders." By F. Russon. (Birmingham: Norman Tiptaft Ltd. 1950. Price 10s.)

Slabs Spanning in Two Directions Analysed by Consideration of Pattern of Fractures.

By Professor H. CRAEMER (Alexandria).

The application of the plastic theory to the analysis, by consideration of the pattern formed by the fractures, of slabs spanning in two directions is described in the following.

Deformations, and especially changes of curvature, of those parts of a slab in which the stresses are well below the yield stresses can be assumed with sufficient accuracy to be exceedingly small compared with the deformations of those parts where fractures occur. The fragments between the lines of fracture may for practical purposes therefore be considered to be planes, and lines of fracture are consequently straight. These assumptions are confirmed by tests showing

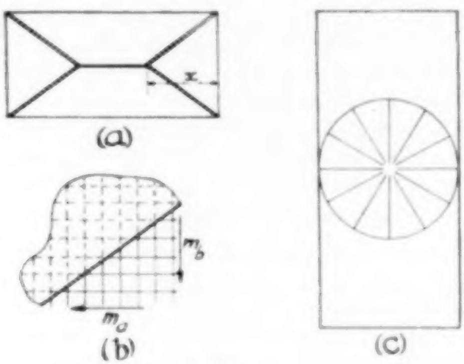


Fig. 1.

the behaviour of rectangular slabs carrying uniformly-distributed loads, the fracture diagrams being as shown in *Fig. 1a*. Only in such cases as circular slabs or where concentrated loads occur are the fragments of a fractured slab exceedingly small, in which cases a network of cracks as shown in *Fig. 1c* results. Reinforcement crossing the lines of fracture is assumed to be subjected to the same stress throughout, the stress being the yield-point stress. Where reinforcement crosses a line of fracture at right-angles the line forms the axis of rotation of the fracture moment, which is constant along the line of fracture because the same stress occurs throughout. This simplifies the calculations. Where, however, a line of fracture is crossed obliquely by reinforcement bars, the axes of rotation are at right-angles to the bars, as indicated by the vector arrows in *Fig. 1b*, in which case the components m_a and m_b of the fracture moment are constant along the line of fracture. Since the moment along the lines of fracture is the greatest moment, there are no shearing forces at the fractures, except in some cases where several fractures intersect or where a free edge is met obliquely by a line of fracture.

Where the pattern of the fracture is known, the fracture moment is best

determined by imparting to the system a virtual displacement so that the fragments remain plane and rotate against each other along the lines of the fracture without causing relative displacements, that is if the system is displaced as actually happens at failure. Internal work A_i is then performed by the fracture moments alone. The sum of the internal work and the work A_e performed by the external load is zero, that is $A_i + A_e = 0$, an expression which can be evaluated since the fracture moment is included in A_i.

If the fracture diagram (*Fig.* 1*a*) contains one or more unknown elements $x_1, x_2, \ldots x_n$, the form of the equation for the fracture moment is $m = f(x_1, x_2, \ldots x_n)$, the correct solution being that in which m is greatest. Therefore

$$\frac{\partial m}{\partial x_1} = \frac{\partial m}{\partial x_2} \ldots = \frac{\partial m}{\partial x_n} = 0 \qquad (1)$$

This method can be demonstrated by applying it to problems which can be solved only laboriously by the theory of elasticity or are insoluble by that theory.

Let slab ABCD (*Fig.* 2*a*), which is loaded with a uniformly-distributed load p, be rigidly fixed along AB and AD and unsupported along BC and CD. The slab is reinforced in the bottom and top in two directions. The reinforcement in the bottom and top parallel to AB is that required to resist fracture moments of m_a and m_a' respectively along AD. For the reinforcement parallel to AD the corresponding moments are m_b and m_b' along AB. The four moments are required to satisfy any arbitrary proportions, such as $m_a : m_a' : m_b : m_b' = 1 : 3 : 1 : 2$, or

$$m_a = m_b,\ m_a' = 3m_a,\ m_b' = 2m_a \qquad (2)$$

If m_a is known, the remaining moments and the amount of reinforcement can be calculated. A probable fracture may be along AE across which a positive moment occurs while negative moments occur along AB and AD. The distance DE $(= x)$ is not known. The load on fragment ADE is $1{\cdot}0\,xp$. If unit virtual displacement downwards is imparted to point E, the centre of gravity of the load is moved a distance of one-third, and the external work performed is $\frac{xp}{3}$. Fragment ABCE can be divided into two parts AFE and BCEF, the loads on which are $1xp$ and $2(3 - x)p$ respectively, the displacements of the centres of gravity being one-third and one-half. Therefore the total work performed by the external forces is given by

$$A_e = \frac{xp}{3} + \frac{xp}{3} + \frac{2(3 - x)}{2}p = \left(3 - \frac{x}{3}\right)p.$$

Fragment AED rotates about axis AD to an angle of $\frac{1}{x}$, and moments m_a and m_a' alone perform internal work, the resultant of which is $2(m_a + m_a')$. The work performed is $-\frac{2(m_a + m_a')}{x}$. Similarly fragment ABCE rotates about AB to an angle of $\frac{1}{2}$ and the resultant of the moments is $xm_b + 3m_b'$. The internal work performed is $-\frac{xm_b + 3m_b'}{2}$. The total internal work A_i is

$$-\frac{2(m_a + m_a')}{x} - \frac{xm_b + 3m_b'}{2}.$$

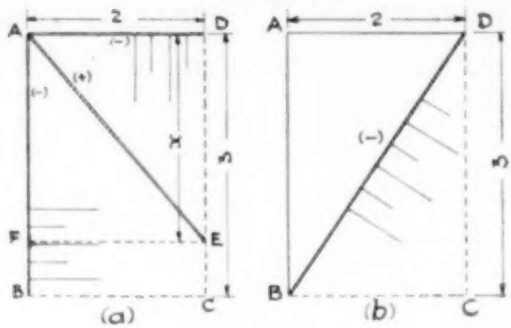


Fig. 2.

Having regard to equations (2),

$$A_i = -\left(\frac{8}{x} + \frac{x+6}{2}\right)m_a.$$

Since $A_i + A_e = 0$,

$$m_a = \frac{18x - 2x^2}{48 + 18x + 3x^2}p \quad . \quad . \quad . \quad . \quad . \quad (3)$$

It follows from equation (1) that

$$(18 - 4x)(48 + 18x + 3x^2) - (18x - 2x^2)(18 + 6x) = 0,$$

the solution of which is $x = 2{\cdot}22$. Therefore $m_a = m_b = 0{\cdot}293p$; $m_a' = 0{\cdot}88p$ and $m_b' = 0{\cdot}585p$.

Another probable fracture diagram is that in *Fig. 2b* wherein the free corner C rotates with a negative moment about the fracture BD. The load on fragment BCD is $3p$, and, with unit virtual displacement of C downwards, the centre of gravity of the load is moved by one-third, therefore $A_e = p$. The components of the fracture moment along BD are $2m_a'$ and $3m_b'$. The rotation of the fragment about BD can be considered in two parts, namely a rotation to an angle of one-third about axis AD and a rotation to one-half about axis AB, the interior work performed therefore being

$$A_i = -\left(\frac{2}{3}m_a' + \frac{3}{2}m_b'\right).$$

Having regard to equations (2), $A_i = -5m_a$. Substituting in $A_i + A_e = 0$ and reducing, $m_a = 0{\cdot}2p$, which is a smaller moment than that calculated on the assumption shown in *Fig. 2a* ; therefore in this example the fracture in *Fig. 2b* will not occur.

If it is assumed that the fracture in *Fig. 2a* is at 45 deg., $x = 2$, and, substituting in (3), $m_a = 0{\cdot}2915p$. The foregoing shows that if a reasonable fracture diagram is assumed the moments can be calculated fairly accurately without having to consider the condition represented by equations (1) to obtain the maximum values.

A problem which can hardly be solved by other methods is that of a triangular slab (*Fig.* 3) fixed along sides AB and BC, and unsupported along AC. Assume that the slab carries a load P uniformly distributed along AC. Reinforcement

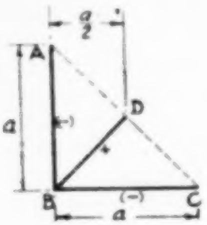


Fig. 3.

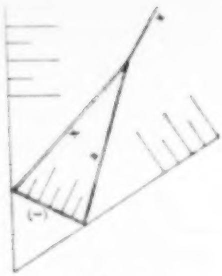


Fig. 4.

is provided in the top and bottom so that the positive and negative fracture moments m are equal. Due to symmetry, a fracture diagram as in *Fig.* 3 applies. If unit virtual displacement downwards is imparted to D, the centres of gravity of the loads $\frac{P}{2}$ on each of the two fragments ABD and CBD is lowered by $\frac{1}{3}$, so that $A_e = \frac{P}{2}$. The resultant of the positive moments along the fracture line BD, considering fragment BCD, is 0·5 am, rotating about BC. The fracture along BC contributes a moment of am in the same sense, giving a total of 1·5 am. The fragment rotates about BC at an angle $\frac{2}{a}$, so that the internal work performed is $-3m$ or, for both fragments, $A_i = -6m$. Equating internal and external work, $m = \frac{P}{12}$. Therefore the fracture moment is independent of the span.

In slabs of this nature, fracture may also occur in a system of forked lines as in *Fig.* 4. A square slab fixed along four edges and carrying a uniformly-distributed load produces, in this manner, a fracture moment which is about 10 per cent. greater, but the effect is less for rectangular slabs, and non-existent for freely supported slabs. For slabs with acute angles, and for slabs carrying concentrated loads, diagrams with forked fractures give higher fracture moments than if a simple fracture pattern is assumed.

Consideration of the lines of fracture of reinforced concrete slabs is also given by K. W. Johansen in the final report of the third congress of the International Association for Bridge and Structural Engineering.

Reinforced Concrete Residential Flats at Pimlico, London.

On a riverside site at Pimlico, London, eleven blocks of residential flats are in course of construction for the Westminster City Council. Block No. 1 (*Fig.* 1) is now occupied, Nos. 2 and 5 are almost complete, and No. 6 is in course of construction. Construction of blocks Nos. 7, 8, 9, and 10 started recently but work has not yet commenced on Nos. 11, 12, and 14. Blocks 1, 2, 5, and 6, which are described in this article, are of reinforced concrete construction; each comprises nine stories, a basement, and cylindrical tank rooms above the roof. Block No. 7 will be seven stories high and is a reinforced concrete frame structure. Nos. 8, 9, and 10 will be each four stories high and constructed with load-bearing brick walls and reinforced concrete floors.

The reinforced concrete structure of Blocks Nos. 1, 2, 5, and 6 comprises load-bearing walls, wall-columns, and solid floor and roof slabs. The story-height is 9 ft. 4 in. (floor to floor) and the width is about 30 ft. Block No. 2 is 282 ft. long, and Nos. 5 and 6 are each 340 ft. long, but block No. 1 is 423 ft. long as it extends over a "way-through" as shown in the longitudinal section in *Fig.* 2, which also shows typical cross sections and plans of Block No. 1. (See pages 286 and 287 for *Fig.* 2.)

Figs. 3 and 4 show typical details of some of the reinforcement and *Fig.* 5 to *Fig.* 10 show various stages of the construction. The external walls are faced with blue bricks up to the sills of the windows of the ground floor story and with yellow bricks above. A strip of concrete 4 in. deep is exposed to form the face of a nib above each line of windows carrying the brick skin and windows. The concrete is also exposed on the outer face of the walls of the top story, which is set back, and the tank rooms, on the piers and slabs of the "way-through", and on the faces of the upstand beams of the balconies and on the flanks and back walls of the recessed balconies. Where the exposed concrete is painted it is coated with an enamel which has a chlorinated rubber base producing a glazed surface which can be washed.

Fig. 1.—Rear Elevation of Block No. 1.
The small building in the foreground is a refuse-collecting shed.

The buildings are designed for the loads

and wind pressures recommended in British Standard Code of Practice CP 3, Chapter V, Loading. The stresses and design of the reinforced concrete work are in accordance with British Standard Code CP 114 (1948). The proportions of the structural concrete are generally 1 : 1·8 : 3·6 using ordinary Portland cement, crushed gravel, and washed pit-sand. The crushing strengths specified are 2200 lb. per square inch at seven days or 3300 lb. per square inch at 28 days. Cubes are tested at seven days, and at 28 days only if the strength at seven days is not sufficient. Generally the strengths at seven days are from 2500 lb. to 4000 lb. per square inch. The slumps vary from 1 in. upwards, the maximum slumps specified being 3 in. for slabs and 4 in. for walls. The concrete is consolidated by immersion vibrators.

For block No. 5 the concrete was distributed from the mixer to the point of placing by a pump, but for the other blocks the concrete is discharged from the mixer into bottom-opening skips which are lifted by derrick cranes. The cranes (*Fig.* 6) travel on short tracks between the blocks and have jibs 120 ft. long, and are also used to lift the panels of shuttering and skeletons of reinforcement. The average rate of progress of each block is about one story a month.

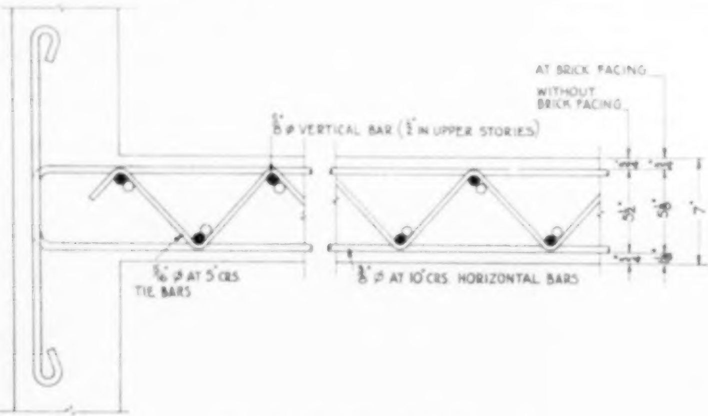


Fig. 3.—Details of Walls.

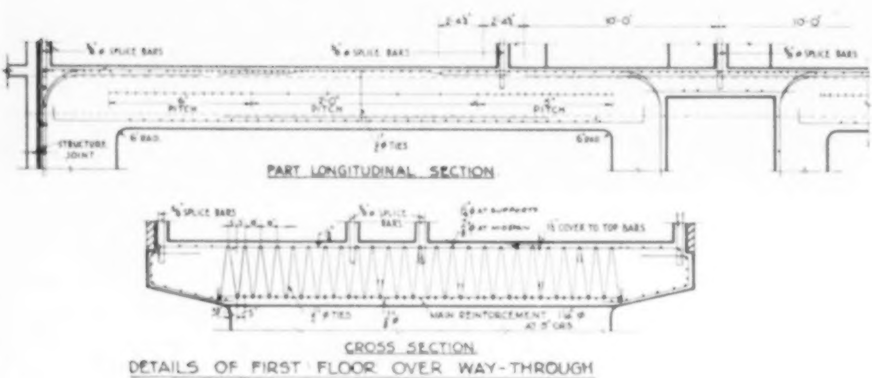


Fig. 4.

Construction of the first block commenced in August, 1948.

Foundations and Basement.

The foundations, which are of plain concrete, bear on ballast which underlies filling and clay at a depth of about 15 ft. The load on the ballast does not exceed 4 tons per square foot. Walls and piers of plain concrete extend from the ballast to within 3 ft. 2 in. of the lowest part of the basement floor (*Fig.* 2), and at the level of the top of the foundation walls 3 in. of plain concrete are spread over the site. Waterproofing is laid on the plain concrete and a reinforced concrete slab overlies the waterproofing which is carried up the outside of the basement walls to waterproof completely the work below ground. The basement floor, which is not connected to the walls, is a 5-in. concrete slab, laid to fall, carried on hardcore deposited on the lower reinforced concrete slab.

Walls.

The reinforced concrete walls above the ground floor are generally 7 in. thick and are lined on the inside with 1-in. wood-wool slabs. The reinforcement in the walls (*Figs.* 3 and 10) is a layer of vertical and horizontal bars near each face, the two layers being maintained in position by $\frac{3}{16}$-in. diagonal tie-bars. The cover of concrete is generally not less than $\frac{3}{4}$ in., but where there is a brick facing the cover is not less than $1\frac{1}{2}$ in., allowance being made for the provision of thin metal slots that are embedded in the concrete and in which metal bonds, projecting horizontally into the joints of the brickwork, are inserted as the bricks are laid. The brickwork is built after completion of the concrete walls and is supported on the 4-in. nib, which projects 5 in. from the wall.

The shuttering for the faces of walls to which brickwork is to be attached is of steel plates (*Fig.* 9) in panels, each of which is the height of one story, and stiffened by a framework of steel scaffold-tubes. Upon removal of the shuttering the outside face of the concrete is coated by bituminous waterproofing applied by spray. The shuttering for the inner face is provided by the wood-wool slabs which are supported by a timber frame and steel scaffold-tubes. Raking tubes secured to steel-wire loops built into the floor slabs hold the inner shuttering in position. The shuttering for the faces of concrete walls and wall-columns, which are not lined with wood-wool or faced with brickwork, but which are to be rendered, is treated with a liquid to retard the setting of the cement on the face so that a key can be formed for the rendering (*Fig.* 5).

The reinforced concrete walls of the cylindrical tank rooms on the roofs of

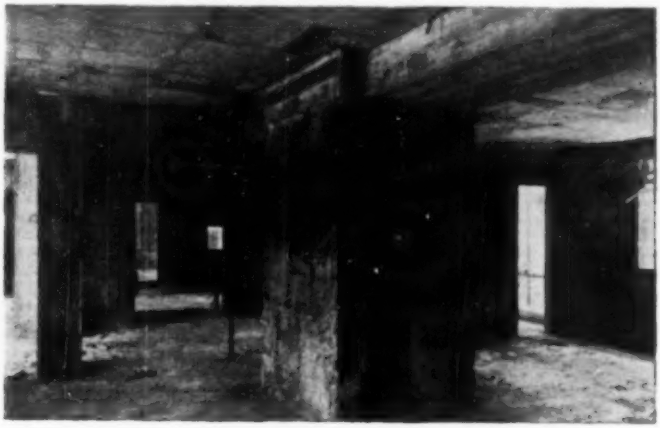


Fig. 5.—Concrete before Application of Finishes.

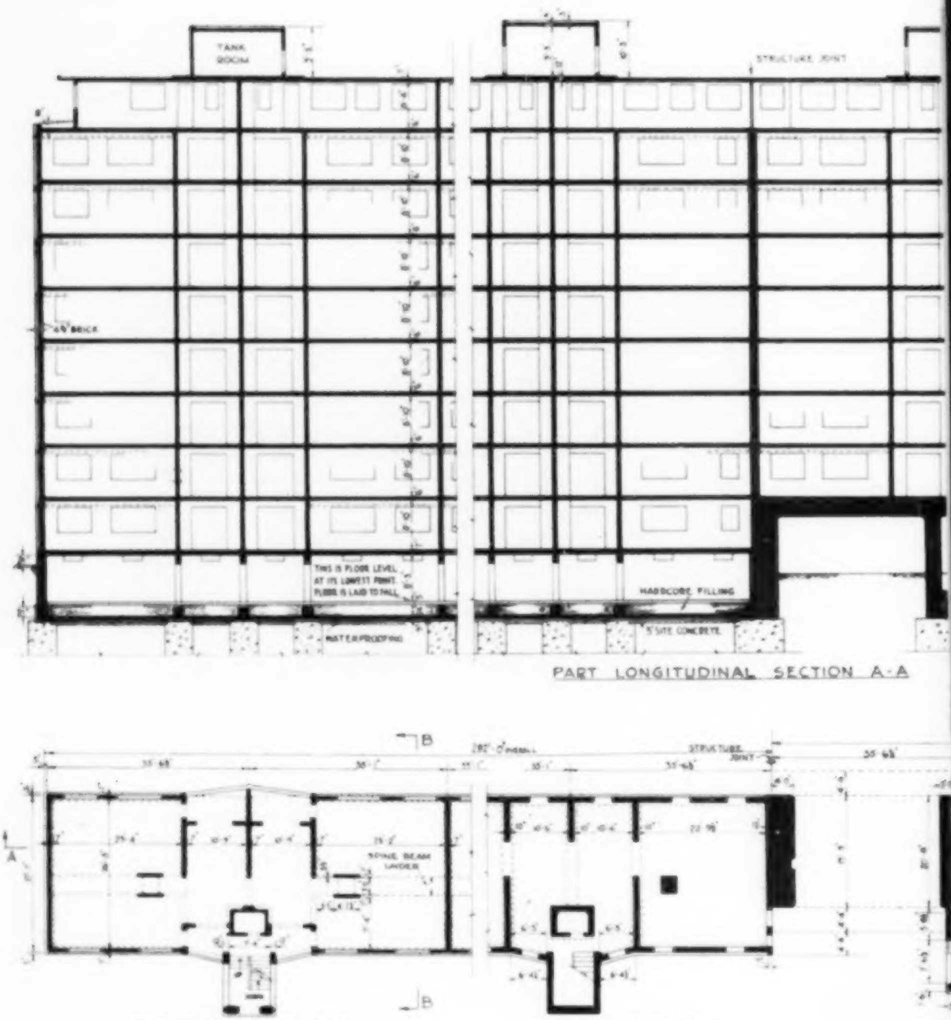
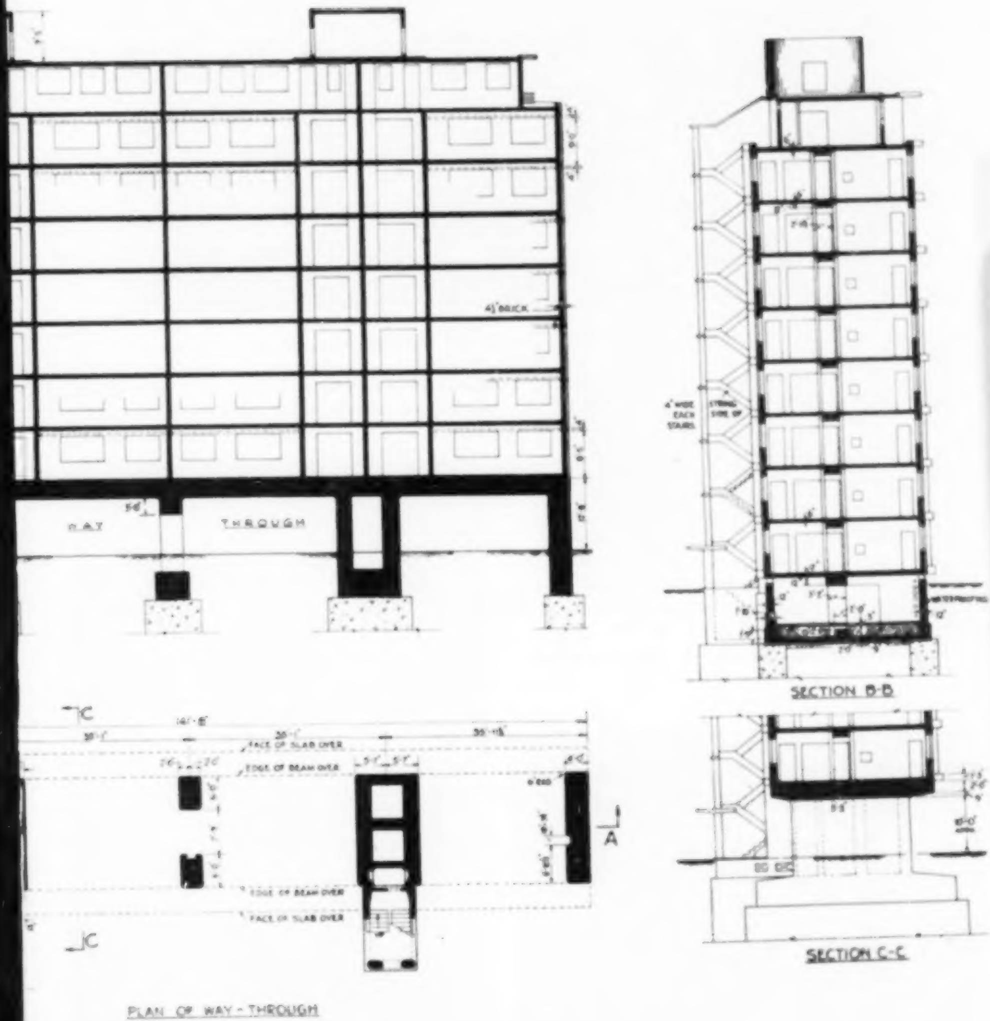


Fig. 2.—Reinforced Concrete F



Pimlico, London : Block No. 1.

Fig. 6.—Arrangement of Travelling Derrick Cranes, Block No. 2.

each block are 5 in. thick and the internal diameter of the tank rooms is 17 ft.

Floors.

The floors generally are 6-in. reinforced concrete slabs spanning transversely between the outer longitudinal walls and a longitudinal spine beam about 3 ft. wide and projecting 12 in. below the bottom of the slab. Longitudinal partition walls forming a central corridor are in line with the sides of this beam, the existence of which is not therefore evident in the rooms or corridors. The shuttering for the soffits of the floor slabs are metal plates supported on adjustable steel props. A length of about 50 ft. of floor of the full width of the block is concreted in one operation, rebated construction joints, either straight across the floor or zig-zagged, being formed at the end of each day's work. The covering

Fig. 7.—Reinforcement in Piers of Way-Through.

Fig. 8.—Supports for Staircase.

of the floors is generally wooden boards on sound-insulating pads. Where floors are covered with a composition finish, a layer of glass-wool is provided between the structural concrete and the screeding, the total thickness of the covering being about 3 in.

Timber shuttering lined with pressed grainless board or sheets of aluminium is used for the cast-in-situ reinforced concrete stairs (*Fig.* 8).

The extension of block No. 1 necessitates that the first floor should span about 30 ft. to provide a "way-through" at ground level. The slab of the first floor is therefore 3 ft. 3 in. thick and is heavily reinforced (*Fig.* 4), and is designed to act monolithically with the supporting piers. The compressive reinforcement is retained by ½-in. lacing ties which are at 24-in. centres near the middle of the span and at 5-in. or 6-in. centres near the supports. The extension is separated completely from the main part of block No. 1 by a structure joint across which flexible copper sealing-strips are provided. There are no other expansion or contraction joints elsewhere in the buildings. The vertical reinforcement in the piers is bent over to project into the first-floor slab. The overhanging bars were temporarily supported on steel scaffolding (*Fig.* 7).

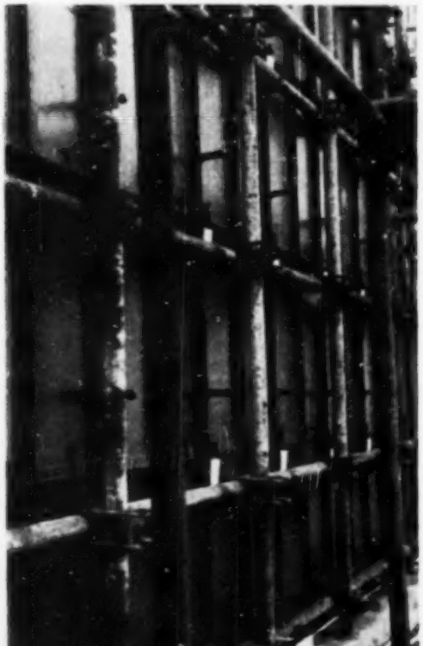


Fig. 9.—Shuttering for Walls.

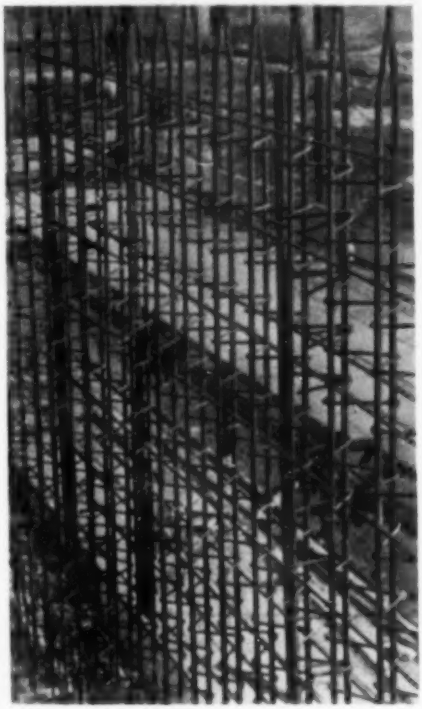


Fig. 10.—Reinforcement for Load-carrying Walls.

Ducts for District Heating.

Hot water is to be supplied to the flats from the power station at Battersea on the opposite side of the river Thames. The water flows and returns in 12-in. steel pipes which pass under the river in a tunnel constructed about 1880 and used by the Metropolitan Water Board. On both sides of the river the pipes are brought to the surface up old shafts about 80 ft. deep, but from the top of the shaft to the site the pipes are in a new underground reinforced concrete duct about 700 ft. long.

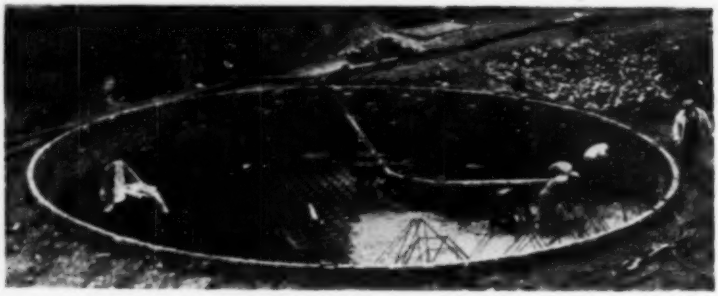


Fig. 11.—Foundation of Tower.

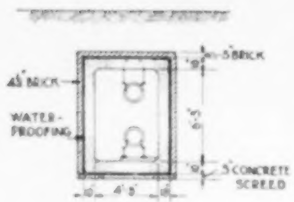


Fig. 12.—Typical Section through District-Heating Duct.

From the shaft the duct proceeds under Grosvenor Road parallel to the river for a distance of about 340 ft. from the shaft and at Antrobus Street turns northwards, passes under a water main, and emerges into the disused Belgrave dock. At the inner end of the dock the duct passes below new workshop buildings to the new pumphouse from which the pipes proceed to an accumulator tank. The steel tank contains 2300 tons of water (up to 200 deg. F.), and is built in a steel tower with glass panels, 35 ft. diameter and 126 ft. high. The lower part of the tower for a height of 30 ft. is encased in a reinforced concrete wall the outer face of which is lined with granite setts. The foundation of the tower is a reinforced concrete raft, circular in plan, seen in course of construction in *Fig.* 11.

The duct (*Fig.* 12) is rectangular in cross section and is 4 ft. 3 in. wide and 6 ft. 3 in. high inside. The walls and bottom and top slabs are 10 in. thick. The level of the top of the duct is from 3 ft. to 9 ft. below the surface, and as the ground is waterlogged it was necessary to provide cofferdams during the construction, which was on the cut-and-cover principle. The trench was therefore lined with temporary sheet-steel piling driven down to clay, but timbering was used near water-mains because the vibration due to piling would have been detrimental to the mains. Before the reinforced concrete slabs and walls of the duct were constructed, a layer of lean concrete 3 in. thick was deposited in the bottom of the trench the sides of which were lined with 4½-in. brick. A bituminous waterproof layer extends over the lean concrete and the inner faces of the brick lining which forms permanent shuttering for the outer faces of the walls. The inner faces and the underside of the top slab were shuttered with steel plates. The waterproof layer is carried over the top of the duct, where it is protected with a 3-in. layer of bricks. In the old dock the duct was constructed in the open. Plain 1 : 5 : 10 concrete was deposited on the gravel at the bottom of the dock up to the level of the underside of the duct. The duct is buried in the earth filling of the dock.

The architects for the scheme are Messrs. Powell and Moya. The structural consulting engineers are Messrs. Scott & Wilson. The consulting engineer for the district heating is Mr. S. B. Donkin. The contractors for blocks Nos. 1, 2, 5, and 6 and for the culvert in connection with the district heating are Messrs. Holloway Brothers (London), Ltd. The contractors for blocks Nos. 7 to 12 and 14 are Messrs. Rowley Bros.

Construction with Moving Forms.—VI.*

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

Reinforcement.

DESIGN.—The reinforcement must be designed with the requirements of moving-form construction in view, and it must be as simple as possible. Vertical bars must be as few as possible. *Fig.* 34 is a plan of some of the walls of a grain silo with rectangular bins of different sizes. At each intersection of walls there are stirrups at the same regular spacing as the horizontal bars in the walls, and therefore indicate the correct spacing for the horizontal reinforcement. In this

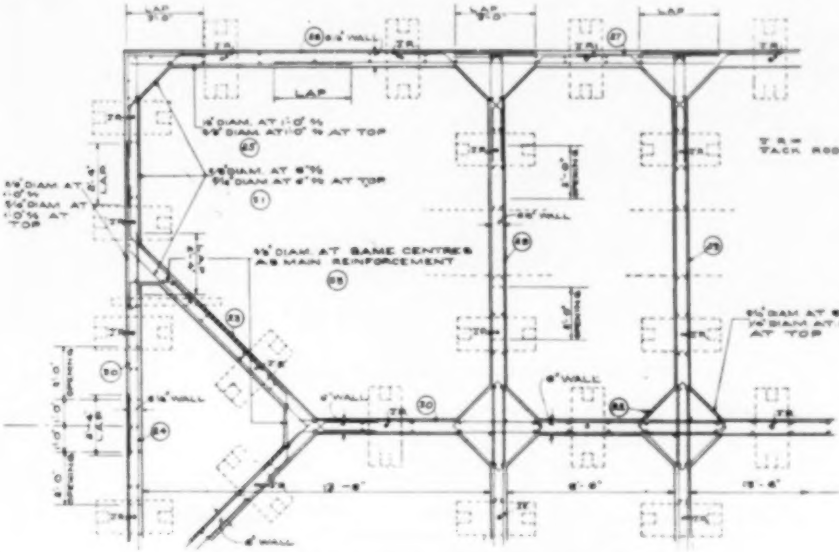


Fig. 34.—Reinforcement in Rectangular Bins.

case some vertical bars are provided in the walls. In a circular bin the horizontal bars in the wall are placed generally in the middle of the wall.

Generally, bars of less than $\frac{3}{8}$-in. diameter are not used, especially if there are no vertical bars, as small bars sag and it is preferable to use bars $\frac{1}{2}$ in. or more in diameter at wider spacing. Despite the use of bars of reasonably large diameter, it is sometimes found that in long walls it is advisable to provide some vertical bars to prevent sagging of the horizontal bars. It may happen, especially in circular bins, that a bar will " spring " after the guide has risen above it; most probably the bar will spring to the face of the wall, and it is difficult to adjust the bar even in the relatively new concrete beneath the bottom of the forms. Vertical bars assist in keeping the horizontal bars in position and prevent sagging.

The provision of hooks at the ends of the bars may be a source of trouble,

* Continued from July, 1950.

especially if the bars turn and the hooks protrude from the wall. In straight walls it is common to provide an extra length of bar for the development of bond. Additional overlapping bars can be provided for this purpose if the length of the wall is too short to accommodate the longer bars. It is usual, however, to provide hooks at the ends of the main bars in circular walls.

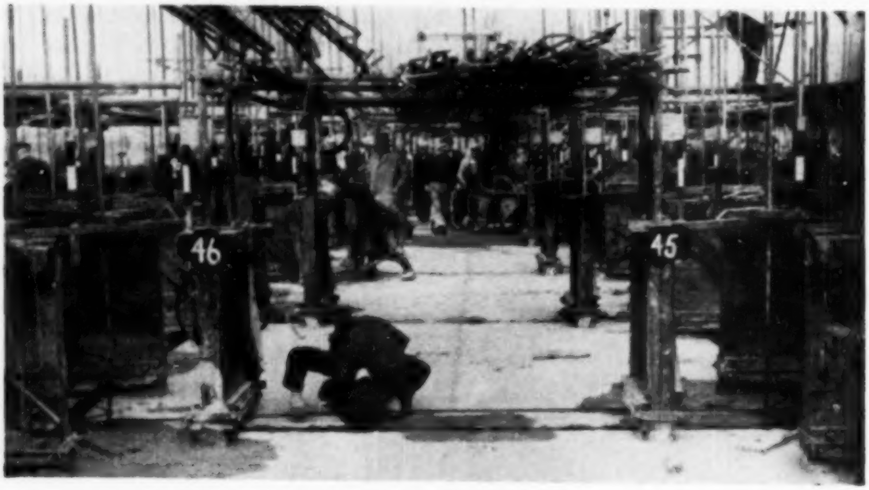


Fig. 35.—Racks for Storing Reinforcement.

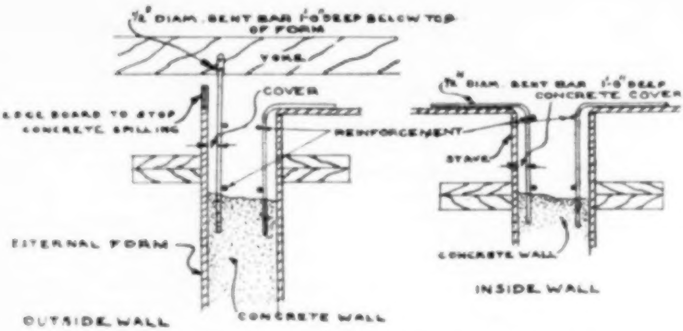
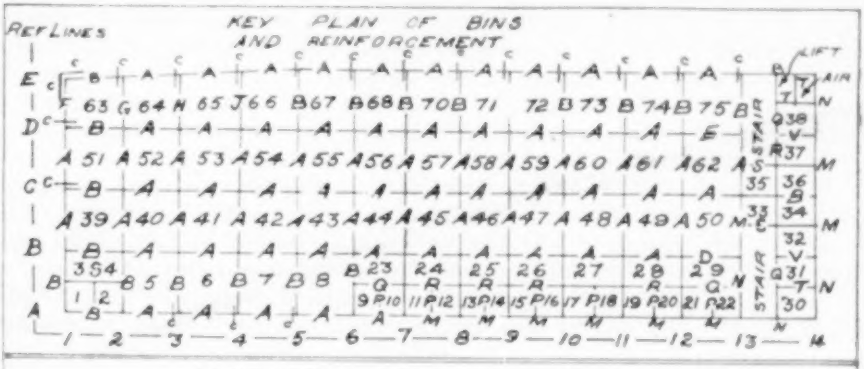


Fig. 36.—Providing Required Cover of Concrete.

Stacking Reinforcement on the Deck.—For stacking the reinforcement, it is an advantage to have horizontal racks on top of the yokes on which to place sufficient bars for one day's work; this ensures that the runways are kept free for the concrete barrows. An arrangement of racks is shown in *Fig.* 35.

Cover of Concrete.—The minimum cover of concrete over the reinforcement in moving form construction should, in the writer's opinion, be 1 in. Less cover is very difficult to maintain in a wall, and if a watertight structure is required on an exposed site it is better to have 1½ in. of cover. If the cover



HORIZONTAL BARS IN WALLS

5/8" DIAM					
INDEX LETTER	LENGTH STRAIGHT	LENGTH BENT	INDEX LETTER	LENGTH STRAIGHT	LENGTH BENT
A	21'-6"	21'-0"	F	15'-0"	14'-6½"
B	19'-6"	19'-0½"	G	16'-0"	15'-8"
C	4'-6"	4'-0½"	H	17'-3"	16'-9½"
D	18'-9"	18'-3"	J	18'-3"	17'-11"
E	20'-6"	[illegible]			

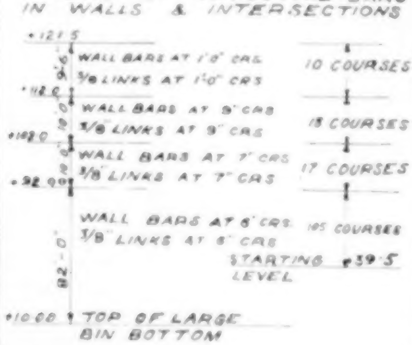
3/8" DIAM					
INDEX LETTER	LENGTH STRAIGHT	LENGTH BENT	INDEX LETTER	LENGTH STRAIGHT	LENGTH BENT
M	21'-3"	21'-0"	S	15'-9"	15'-5"
N	19'-3"	19'-0½"	T	8'-0"	7'-9"
P	8'-3"	7'-11"	V	9'-0"	8'-8¾"
Q	16'-9"	16'-5½"			
R	17'-9"	17'-6"			

VERTICAL BARS AT INTERSECTIONS OF WALLS
1" DIAM

REF LINES	A Nº	A LENGTH	B Nº	B LENGTH	C Nº	C LENGTH	D Nº	D LENGTH	E Nº	E LENGTH
1	6	18'-6"	5	19'-0"	5	19'-0"	5	19'-0"	5	19'-0"
2	5	"	4	"	4	"	4	"	5	"
3	5	"	4	"	4	"	4	"	5	"
4	5	"	4	"	4	"	4	"	5	"
5	5	"	4	"	4	"	4	"	5	"
6	5	"	4	18'-3"	4	"	4	"	5	"
7	5	17'-3"	4	"	4	"	4	"	5	"
8	5	"	4	"	4	"	4	"	5	"
9	5	"	4	"	4	"	4	"	5	"
10	5	17'-3"	4	18'-3"	4	19'-0"	4	19'-0"	5	19'-0"
11	5	"	4	"	4	"	4	"	5	"
12	5	"	4	17'-0"	4	"	4	"	5	"
13	5	"	3	18'-9"	4	"	3	"	5	"
14	6	"	5	17'-0"	5	17'-0"	5	17'-0"	6	"

FOR TOP LIFT — D_1 TO D_{13} – 7'-0"
C_1 TO C_{13} – 12'-0"
E_1 TO E_{13} – 22'-0"

SPACING OF HORIZONTAL BARS IN WALLS & INTERSECTIONS



NUMBER & WEIGHT OF BARS IN ONE COURSE

5/8" DIAM			3/8" DIAM		
BAR	Nº OF PAIRS	FEET	BAR	Nº OF PAIRS	FEET
A	72	3096·0	M	9	382·5
B	28	1092·0	N	4	154·0
C	19	171·0	P	7	115·0
D	1	37·5	Q	4	134·0
E	1	41·0	R	7	248·5
F	1	30·0	S	4	126·0
G	1	32·0	T	3	48·0
H	1	34·5	V	2	36·0
J	1	36·5			
TOTALS	125	4570·5	TOTALS	40	1244·5
WT. CWTS		42·58	WT. CWTS.		4·20

Fig. 37.—Typical Schedule of Reinforcement.

is too small rust may soon stain and disfigure the walls. In the case of a structure 100 ft. or more in height, the penetrating power of wind-swept rain is far greater than at ground level. The advantage of moving-form construction in eliminating joints is lost if the reinforcement is not adequately covered. The provision of horizontal links at the intersection of walls helps in placing correctly the main horizontal bars, but if the reinforcement is too complicated there will be a tendency for the reinforcement to be lifted as the forms are raised.

The reinforcement has to be fixed with the minimum of tying, otherwise this operation retards the work. It is necessary to maintain the required cover of concrete, one method of achieving this being to provide steel-pin guides bent down from the top of the forms, the distance from the outside of the pin to the inner face of the form being equal to the required cover as on the left-hand side of *Fig.* 36. The pins are generally about 1 ft. 3 in. long. The reinforcement

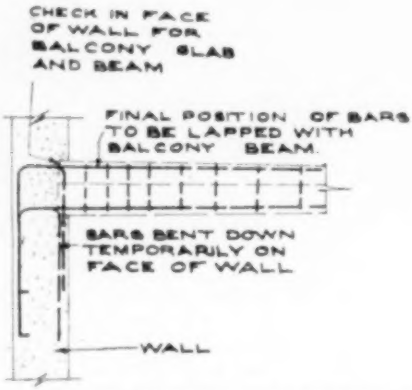


Fig. 38.—Arrangement of Reinforcement at Projections from Wall.

is placed at the top of the forms as they rise, and is guided into position ; by the time the initial set of the concrete has taken place the reinforcement is firmly fixed in the correct place. An alternative method of providing the correct cover by the use of steel guides is shown on the right-hand side of *Fig.* 36.

Bar Schedules.—Great care must be taken to follow the drawings when placing the reinforcement. The employment of first-class steel-fixers is essential, as the work requires expedition and care and the ability to work by memory. To facilitate fixing, it is advantageous for diagrams to be made so that the steel fixing is shown at a glance. *Fig.* 37 shows typical notes and tables for this purpose, and includes a key plan, schedules of bent and straight bars in the walls and intersections of the walls, and the spacing of the horizontal bars in the walls and intersections. *Fig.* 37 also shows the number and weight of horizontal bars in one course (or layer) in the walls. In the walls of the structure for which these schedules were prepared there were no vertical bars except at the intersections.

Slabs Projecting from Walls.—It may be necessary to begin moving the forms at a level lower than that at which some projecting feature occurs ; for

example, a cantilevered balcony may project from the wall of the main structure and in this case it is essential for the bars in the balcony to be embedded downwards into the wall or pilaster. *Fig.* 38 shows how this can be achieved. Bars for the balcony are placed correctly in the wall or pilaster, but the projecting part of the bars is bent downwards along the face of the forms. When the moving forms have passed these bars, the bars are bent up in their correct position. If dowel-bars are provided to project from the wall or pilaster and to lap with the main bars in the balcony, it is an advantage if the two sets of bars are welded together.

In the case of suspended floors, the floor reinforcement cannot extend into the walls, but dowel-bars must be provided the outer ends of which are temporarily bent horizontally or vertically against the forms. After the forms have passed the level of the slab, the dowel-bars are straightened and the floor reinforcement placed in position to overlap with them.

Fixing Flat-bottom Rails to Concrete Sleepers.

In a proposal by Professor Hartmann for fixing flat-bottom rails to reinforced concrete sleepers (described in "Der Eisenbahnbau" and reproduced in a recent number of "Betonstein-Zeitung"), direct connection between the rail and sleeper is avoided, because the transmission of forces through small areas subjected to great intensities of pressure is unsuitable for a plastic material, such as concrete, when dynamically loaded. The rails rest in a groove between two lugs on the sleepers (*Fig.* 1), the space between the rail and the outer lug being packed with hardwood. In the other space two steel wedges are drawn together and tightened by a bolt (*Fig.* 2) passing through them and thereby securing the rail against lateral movement. An undercut in the outer lug prevents vertical movement. A layer of poplar wood provides a cushion below the bottom flange of the rail and between the steel wedges and the concrete. There are no holes in the concrete to produce weakness of the sleepers.

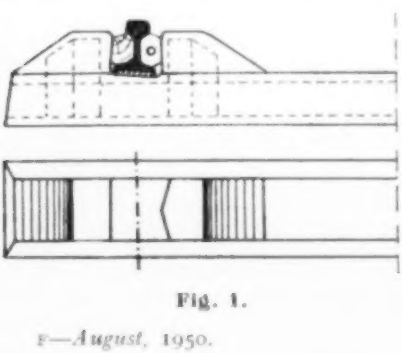


Fig. 1.

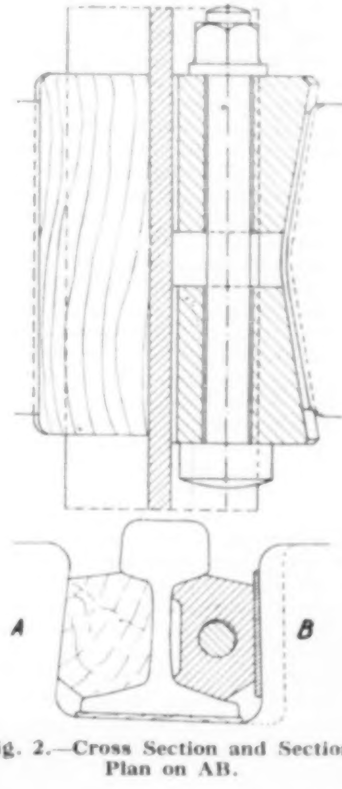


Fig. 2.—Cross Section and Sectional Plan on AB.

A Test of a Prestressed Concrete Railway Bridge Girder.

A PRESTRESSED concrete beam of 43-ft. 6-in. span, and identical in design with those incorporated in an under-line bridge at Normanby Park steelworks of Messrs. John Lysaght, Ltd., was tested to destruction recently. The bridge comprises fourteen precast prestressed concrete beams laid side by side and each 48 ft. long and weighing about 5½ tons. Concrete cast in situ between and over the beams is lightly reinforced. The only shuttering required on the site was on the four vertical faces of the bridge. All the beams were placed in position in one day. It is believed that these are the longest precast prestressed beams in the country.

The purpose of the test (*Fig.* 1), which was carried out by Mr. R. H. Harry Stanger at Elstree, was to determine whether the assumptions of the design regarding safety against cracking, co-operation between the precast beams and the in-situ concrete, and safety against failure were correct.

The dimensions of the beam and the arrangement of the load are shown in *Fig.* 2. The calculated stresses at various stages of loading are shown in *Fig.* 3. The measured deflection, the dead and live loads for which the beam was designed, and the maximum widths of cracks are shown in *Fig.* 2. In the bottom of the beam there were 64 wires and in the top 16 wires of 0·2 in. diameter and having a tensile strength of 100 tons per square inch. The precast beam was steam-cured for five days after casting, and the stretching force of 4700 lb. per wire (equal to a stress of 150,000 lb. per square inch) was released five days after casting. The change of strain upon release was measured at two places and, from these observations and the stress-strain diagram of the wires, the calculated reduction of prestress due to the elastic contraction of the concrete was 11,700 lb. per square inch, that is, about 8 per cent. of the initial prestress. The maximum stress in the concrete was 2610 lb. per square inch. The average strength of concrete cubes was 6400 lb. per square inch.

The in-situ concrete was cast around the precast beam when the latter was sixteen days old. The composite beam, the weight of which was 9·7 tons, was loaded up to 40 tons when the in-situ concrete was 13 days old and had a cube-strength of 3700 lb. per square inch. The moduli of rupture of two prisms of the precast concrete, 4 in. square by 24 in. long, were 830 lb. and 1030 lb. per square inch. Measurements made on prisms subjected to conditions similar to those of the beam showed that no reduction of prestress due to shrinkage occurred.

Fig. 1.—Beam under Test.

The loss due to creep is estimated to be 200 lb. per square inch, that is about 7½ per cent. of the stress in the concrete at the time of release. With this assumption the modulus of rupture of the precast concrete was 1000 lb. per square inch, which is likely to be accurate within ± 10 per cent.

The beam was loaded in increments of 4 tons. During the first loading, the first cracks, having a maximum width of 0·003 in., appeared when the load was 32 tons, but from the load-deflection diagram the actual cracking-load can be assumed to be 29 tons, compared with the design load of 9·2 tons. When the cast in-situ concrete was 25 days old, the beam was loaded for the second and third times. At this time the average cube-strength of the precast concrete was 7480 lb. per square inch and of the in-situ concrete 4150 lb. per square inch. The prism-strengths were 6520 lb. and 3105 lb. per square inch respectively.

At the second loading, during which the load was again increased by increments of 4 tons, no cracks 0·003 in. wide became visible at 20 tons. According to calculation cracks were expected at about 17 tons. The load was then increased to 28 tons when the maximum width of crack was 0·008 in. and the deflection 1·655 in. After unloading, the deflection was 0·003 in.

loading. The load of 42 tons (third loading) was maintained for forty-nine minutes, during which period the beam continued to deflect.

At 43 tons the beam began to yield as a whole. The load was further increased to 44 tons, which caused the beam to fail gradually. Failure (*Fig.* 4) which occurred near one of the loading points and was apparent by the crushing of the

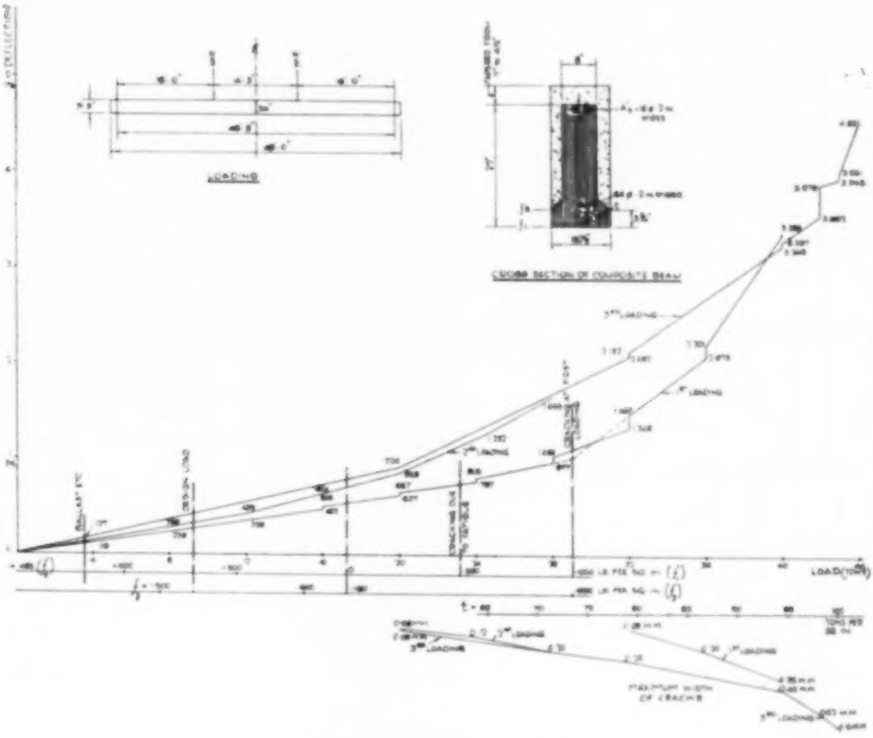


Fig. 2.—Details of the Test.

At the third loading, the load was increased to 20 tons, to 32 tons, and to 40 tons, and was then increased by increments of 1 ton. The deflection at 40 tons was slightly less than that under the same load at the first loading, but this was probably due to the fact that, at the first loading, 40 tons was reached in 3 hours 39 minutes, while at the third loading it was reached in 1 hour 15 minutes. Consequently the effect of creep was much greater at the first

in-situ concrete at the top of the beam, was due to excessive elongation of the wires in the bottom. The stress in the wires was then about the guaranteed minimum tensile strength of 100 tons per square inch. Upon further increase of the deflection at a reduced load, a horizontal crack (*Fig.* 4) appeared at the top of the precast part of the beam, but this was a secondary failure occurring after the in-situ concrete was crushed. No damage was done to the concrete in

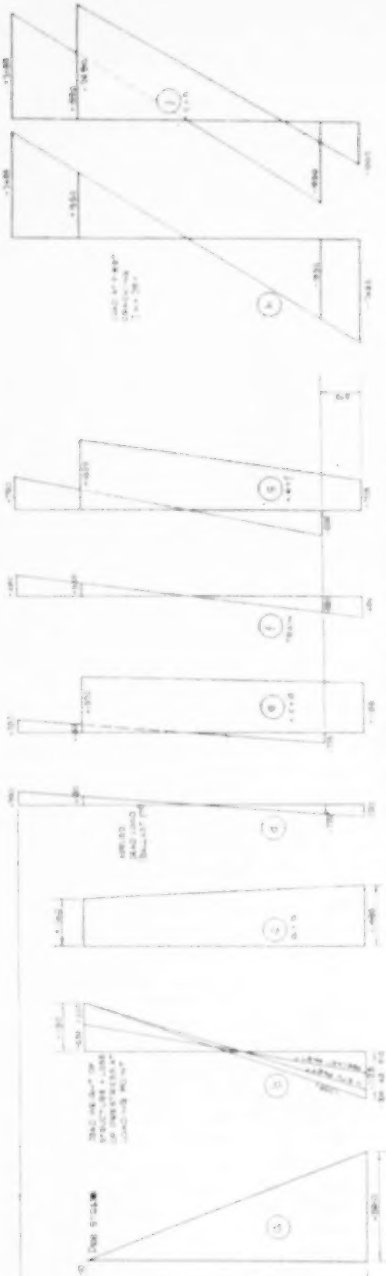


Fig. 3.—Stress Distribution at Various Stages.

the precast part of the beam. The beam was still capable of carrying a substantial load.

Measurements of the strain of the concrete and the wires, made during the first and third loading, indicate that at an early stage of the loading the neutral axis was at the mid-depth of the beam and that it moved gradually upwards as the load increased. At 40 tons the depth to the neutral axis was 13 in. at the first loading and 12 in. at the third loading. At 43 tons the depth was 11 in. The greatest strain measured was 0·0019 which means that the concrete was in a plastic state down to about 5 in. from the top. At the top of the precast beam the strain was 0·00065, and below this level the precast beam contributed more to the compressive resistance than the in-situ concrete because of its much greater strength. It is therefore a close approximation to assume a rectangular distribution of stress on a depth of 11 in. The lever arm was therefore $33{\cdot}3 - 3{\cdot}3 - 5{\cdot}5 = 24{\cdot}5$ in. The maximum bending moment at failure was 120,000 ft.-lb. due to dead load and 788,000 ft.-lb. due to the applied load, the total bending moment being 908,000 ft.-lb. Therefore the total tensile force, which equalled the total compressive force, was $\frac{908{,}000 \times 12}{24{\cdot}5} = 445{,}000$ lb., of which the two ⅞-in. bars provided in the in-situ concrete resisted about $0{\cdot}22 \times 40{,}000$, say, 8800 lb. The contribution of the prestressed wires, with a cross-sectional area of 0·5 sq. in. and placed 3 in. above the neutral axis, where the measured strain was 0·0004, amounted to about $0{\cdot}5 \times 28 \times 10^6 \times 0{\cdot}0004 = 5600$ lb. The stress in the concrete was therefore about $\frac{445{,}000 - 8800 - 5600}{13{\cdot}5 \times 11} = 2900$ lb. per square inch, which was about 93 per cent. of the prism-strength of the top in-situ concrete. The stress in the wires was $\frac{445{,}000}{2{\cdot}01} = 222{,}000$ lb. per square inch. The stresses in the wires given in *Fig.* 3 are based on the assumption that the depth to the neutral axis was 13 in., as ascertained at a load of 40 tons at the first loading.

The conclusions drawn from the test are that the modulus of rupture of the precast concrete was calculated, with

close approximation, to be 1000 lb. per square inch. Although at first loading 92 per cent. of the ultimate bending moment at the third loading was applied, the beam completely recovered and cracking at the second loading occurred at a load at which, by calculation, there was no longer any precompression in the concrete. The beam made a complete recovery after the second loading. Although the calculated tensile stresses in the in-situ concrete were large, no cracks could be seen in this part before cracking occurred in the precast beam. There was no change in the width of cracks at the junction of the precast and in-situ concrete. Full co-operation between the precast prestressed and in-situ concrete was obtained by thoroughly roughening the surface of the precast beam without having to provide protruding links.

At failure the guaranteed minimum tensile strength of the wire was attained. The test shows that the ultimate load of composite prestressed beams with bonded wires can be predicted very closely from the tensile strength of the wires and the prism-strength of the in-situ concrete, in this case the average compressive stress in the concrete being 93 per cent. of the prism strength.

Although the dead weight of the structure was originally carried solely by the precast beam, it was transferred to the composite structure before failure. After failure the beam was still capable of carrying a substantial load, and upon removal of this load most of the deflection disappeared and cracks between the supports and the loading points completely closed.

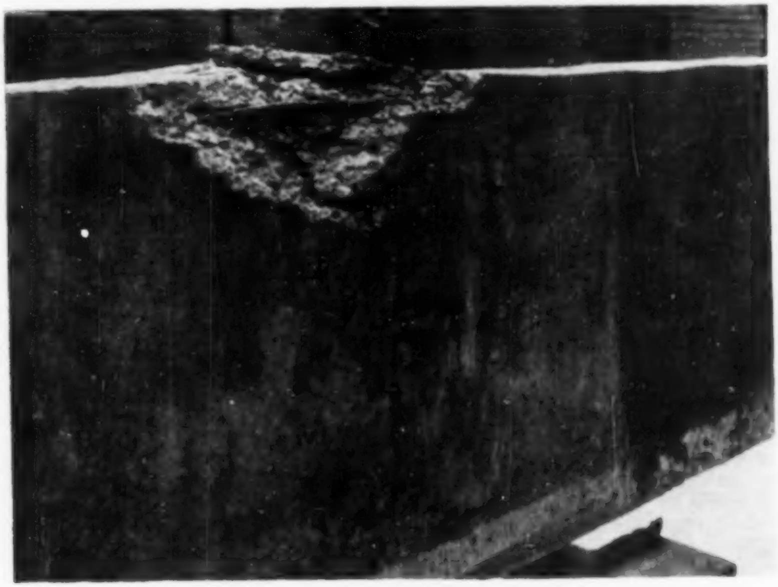


Fig. 4.—Beam after Failure.

The beam was designed by Twisteel Reinforcement, Ltd., and was precast by Messrs. Costain Concrete Co., Ltd. The in-situ concrete was cast on the beam at the testing works.

Variables in Concrete Making.

In our Editorial Note are mentioned some of the points made by Mr. F. N. Sparkes, M.Sc., M.Inst.C.E., in a paper entitled "Control of Variations in Quality of Concrete and its Effect on Mix Proportions," presented to the Reinforced Concrete Association. The paper is given in full in the "Reinforced Concrete Review," October, 1949, and an abstract of the author's recommendations for controlling some of the variable factors in the making of concrete is given in the following.

Cement.—So far as the cement is concerned, the engineer can do little to improve its uniformity. This is a matter for the cement manufacturers, and one can rest assured that if greater uniformity could have been economically achieved the necessary action would already have been taken. It is possible that cement made at any particular works may be more uniform than is indicated by some test results, but there are no published data to show by how much the uniformity may be improved. At present it is doubtful whether the manufacturers could consider such a possibility, but with the return to normal conditions it may become desirable, in the interests of more uniform concrete, to obtain the cement for the whole of a constructional works from one cement factory, as was sometimes done before the war.

Aggregate Grading.—Variations in the grading of aggregate may be reduced by ordering material in several sizes and recombining them in the correct proportions at the batching stage. During the construction of the German motor-roads the aggregates were often delivered in seven different sizes or grades, but this is a refinement which would hardly seem to be necessary. In this country it is becoming increasingly common to order the coarse aggregate in two or three sizes, namely, $\frac{3}{4}$ in. to $\frac{3}{8}$ in. and $\frac{3}{8}$ in. to $\frac{3}{16}$ in. for aggregate of $\frac{3}{4}$-in. nominal maximum size, and $1\frac{1}{2}$ in. to $\frac{3}{4}$ in., $\frac{3}{4}$ in. to $\frac{3}{8}$ in., and $\frac{3}{8}$ in. to $\frac{3}{16}$ in. for aggregate of $1\frac{1}{2}$-in. nominal maximum size.

The grading of sand cannot be adjusted so easily. Much can be done to ensure uniformity by regular sieve tests on the sand as delivered, as this will show whether there is any gradual departure from the specified grading. When the construction work is protracted, the sand pit or quarry may be progressively working into or producing a finer or coarser grading and the sieve tests will indicate whether or not this is happening.

Bulking of Fine Aggregate.—Errors due to this cause can be entirely eliminated by batching by weight in preference to volume. If the latter must be used (and it is suggested that it never should be used) the moisture-content-bulking curve should be established by tests on the sand being used and corrections made to the volume of the gauge boxes as the moisture in the sand changes. Alternatively—and this is the least acceptable—the gauge boxes should be increased in volume by 25 per cent. over that required for dry sand, so that an average allowance is automatically made for bulking.

Batching.—Batching by weight will eliminate the high variations normally associated with batching by volume. Large storage hoppers, filled by dragline buckets or bucket-elevators and elaborate devices for weighing are economical and generally used on large-scale works such as airports and major road works. The principles of weigh-batching can be applied just as easily on works requiring small amounts of concrete where a 10 or 7 cu. ft. mixer is used. When engineers and contractors begin to use weigh-batching they continue to use it in preference to volume-batching, because it is cheaper, more convenient, and more accurate. In the absence of specially designed plant, much can be done with a spring-balance and lever. Where volume-batching must be employed, the gauge-boxes should be deep and of small cross section to avoid excessive errors arising from filling; allowance should also be made for bulking of the fine aggregate.

Compaction.—There is no reason why adequate compaction should not be assured on all work and in all circumstances in view of the variety of compacting devices now available. The proportions and water-cement ratio should be arranged, having regard to the methods of compaction to be used, to ensure good compaction with no segregation and no laitance.

Handling.—The elimination of segregation, which sometimes occurs when concrete is transferred from mixer to dumper,

barrow, or hoist, and from these to its final position, is usually a matter of some difficulty. In many cases baffle plates in the transporting sequence are useful.

The Water-Cement Ratio.—The accurate control of the water-cement ratio requires the use of a laboratory and a technician able to determine the moisture contents of fine and coarse aggregates and to apply the results to the adjustment of the quantities of the several constituents. Whether or not such facilities are available at the site, certain proved methods of handling and storing the aggregate should be adopted to assist in the maintenance of a constant water-cement ratio. The use of stock-piles, sufficiently large to ensure that all the aggregates have been stored at the site at least 24 hours before use, will help to even out the moisture content in the pile. The mixer driver is then more easily able to control the workability of the concrete. The stock-piles should be flat-topped and not conical. It is a good plan to reserve the bottom 1 ft. to 2 ft. of the stock-pile for drainage purposes.

Standard Requirements for Railway Bridges.

THE new edition of " Requirements for Passenger Lines and Recommendations for Goods Lines of the Minister of Transport in Regard to Railway Construction and Operation " (London : H.M. Stationery Office. 1950. Price 1*s.* 6*d.*) requires that, for cast-in-situ reinforced concrete, the working tensile stress in steel reinforcement should not exceed 18,000 lb. per square inch and in precast work it should not exceed 20,000 lb. per square inch. For prestressed concrete the working stresses in the steel wires may be higher, depending on the character of the steel and its anchorage or adhesion in the concrete. Adequate thickness of concrete cover should be provided to the reinforcement, special precautions being taken where necessary on electrified lines to avoid electrolytic corrosion. The structural clearances are generally in accordance with the requirements of the edition of 1928.

The Reinforced Concrete Association.

THE following notes are from the report of the Reinforced Concrete Association for the year 1949.

MEDAL AWARD.—The Council awarded the R.C.A. bronze medal for 1948 to Mr. H. G. Cousins, B.Sc., M.Inst.C.E., for his paper on "Shell Concrete Construction."

CERTIFICATION OF CONCRETING FOREMEN.—The suggestion made by Mr. J. Singleton-Green that the Association should develop means of ensuring the employment of qualified men on reinforced concrete work was considered. It was agreed that the intention was sound, and that an endeavour should be made to implement it, but that the creation of a new class of craftsmen, which would need consideration of labour rates and working rules, was undesirable and unnecessary. It was agreed that the Association's efforts should be directed rather to the training and employment of foremen, charge-hands, and gangers, and that the term "concrete craftsmen" should be avoided. It was also decided that, subject to the concurrence of the Federation of Civil Engineering Contractors and the National Federation of Building Trades Employers, the Association should, as a step towards the employment of qualified men, award certificates to two grades of foremen. This proposal was not approved by the Federations, and the Association has reluctantly abandoned it.

ECONOMY IN THE USE OF STEEL.—A suggestion submitted to the Association by the Steel Economy Committee that steel might be saved by the substitution of the ultimate-load theory for the existing basis of reinforced concrete design was considered. The Committee was advised that such a step was unlikely to have any appreciable effect. A further suggestion that the working stresses in steel of all grades might be increased was also considered, but the Council decided that the stresses permitted by the Code of Practice (CP 114) could not safely be exceeded.

Mr. A. R. Neelands (Cementation Co., Ltd.) has been elected President for the year 1950–51.

SGB

—the originators of tubular scaffolding!

NOW OFFER

WICKHAM PATENT LIFT & FORCE DIAPHRAGM PUMPS

for immediate hire or sale!

- ★ *Delivery head up to 40 feet*
- ★ *Suction lift up to 26 feet*
- ★ *No stuffing glands*
- ★ *Odourless*

S.G.B.

HIRE PLANT

is maintained in first class condition by highly skilled mechanics, thus ensuring trouble-free running whilst in operation.

Is as exclusive in design as it is in performance, it achieves more work for less effort, has a lower cost of maintenance and a longer effective life—*BECAUSE* the suction and discharge efforts are now shared equally between *TWO* diaphragms while the pump is effectively employed on *BOTH* the upward and the downward strokes.

SCAFFOLDING (GREAT BRITAIN) LTD

Plant Division:

MITCHAM, SURREY

Telephone: MITcham 3400 (18 lines)

Telegrams: SCAFCO, MITCHAM

Branches at: ABERDEEN · BIRMINGHAM · BOURNEMOUTH · BRIGHTON · BRISTOL · CAMBRIDGE · CARDIFF · DOVER · DUBLIN · DUNDEE · EDINBURGH · EXETER · GLASGOW · HULL · LEEDS · LIVERPOOL · MANCHESTER · NEWCASTLE · NOTTINGHAM · OXFORD · PLYMOUTH · PORTSMOUTH · SOUTHAMPTON · STOKE-ON-TRENT · SWANSEA

SITUATION VACANT. Surveyor-taker-off required for London office (adjacent King's Cross Station) of reinforced concrete engineers specialising in hollow-tile type of floor construction. Write details of experience, qualifications, and salary required to Box 140, ALLARDYCE PALMER, LTD., 109 Kingsway, London, W.C.2.

THE LIVERSEDGE REINFORCED CONCRETE ENGINEERING COMPANY LIMITED, LONDON

require immediately the following additional staff. Permanent positions and five day week.

(a) Reinforced concrete designers, preferably with experience of project work. Salary £750–900 according to experience, etc.

(b) Designer-detailers and draughtsmen, to work under supervision. Salary according to experience and capabilities.

Applications in confidence, stating details of experience, qualifications and age to 31a John Adam Street, Adelphi, London, W.C.2.

SITUATION WANTED.

SITUATION WANTED. Reinforced concrete and other design and detail. Expert advice available for full or part-time. Address: "Chartered Structural," c/o Winteco Ltd., 18–20 York Buildings, London, W.C.2.

WANTED.

WANTED. Twelve Trillor vibrators in good condition. State price and where lying. TARSLAG, LTD., 117 Dunstall Road, Wolverhampton.

FOR SALE.

Complete concrete steam-curing plant for sale. Batch weigher and overhead bins. Conveyors, elevators, etc. Two "Liner" mixers. 200 bogies and rails. Traverse bogies. Steel doors for kilns. Steam winch, piping, etc. Box 2408, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

STEEL PIPING for sale. 5,000 feet of 3-in., 5,000 feet of 6-in., and 2,000 feet of 8-in. diameter, all screwed and socketted. 5,000 feet of 8-in. and 2,000 feet of 10-in. plain ends. Other sizes in steel and galvanised piping available for immediate delivery. Telephone: Mayfair 9651 (5 lines).

TESTING APPARATUS

MOISTURE TESTING.—Famous Engineers write about "SPEEDY" MOISTURE TESTER: "Not only 'speedy' but can be used satisfactorily by unskilled labour during tests." Accurate; portable; non-electric; inexpensive; used extensively for rapid determination of water-content of concrete aggregates, clays and soils. Write for full particulars to (DEPT. CCE/6) THOS. ASHWORTH & CO., LTD., Burnley, Lancs.

A Patent Reinforcement.

IN reinforced concrete structures there are provided in place of or together with ordinary reinforcing bars, flexible concrete bars containing tensioned reinforcements. The concrete bars are formed by a steel rod or rods being placed under initial tension and concrete cast around them, the reinforcement being placed around the no-stress plane passing through the centre of gravity of the concrete bar to avoid the creation of internal bending stresses. The concrete bar may include both tensioned and untensioned reinforcements.—No. 599.774. E. J. Smedegaard. Oct. 17, 1945.

Road Congress in Portugal.

THE next congress of the Permanent International Association of Road Congresses is to be held in Lisbon next year, probably in September. The Honorary Secretary of the British Organising Committee is Mr. Thos. S. Sinclair, of the Ministry of Transport, Berkeley Square House, London, W.1.

Tower Built with Moving Forms.

ONE of eight television towers erected between Philadelphia and Pittsburgh is 126 ft. high, and the walls, which are 12 in. thick and stand on a 4-ft. raft of concrete, were built in seven days with continuously-moving forms. On the first day the wall was constructed to a height of 23 ft.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.).—Washed sand, 17s. Clean shingle: ¾ in., 15s. 2d.; ⅜ in., 17s. 6d. Thames ballast, 15s. 9d. Broken brick, ¾ in., 17s. 6d.

CEMENT (per ton, delivered at Charing Cross).—

Portland cement, 6 tons and upwards, 65s. 1 ton to 6 tons, 70s. Paper bags and non-returnable jute sacks 9s. 6d. extra.

Rapid-hardening Portland, 6s. above ordinary Portland.

"417," 87s. 6d. in 6-ton loads; paper bags 9s. 6d. extra.

Aquacrete, 31s. 6d. above ordinary Portland; paper bags 9s. 6d. extra.

Colorcrete (buff, red, and khaki), in 6-ton loads, 106s. 6d.; paper bags 9s. 6d. extra.

Snowcrete, £11 17s., inc. paper bags.

"Super-Cement," 6 tons and upwards, 96s. 6d. (bags 8s. extra).

High-alumina cement, 1 ton and upwards, 226s. 6d. per ton; paper bags 8s. per ton extra.

Snowcem paint, 56s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel round bars (per cwt.): ⅝ in. to 2½ in., 28s. 2d. 7/16 in. to ½ in., 29s. ⅜ in., 29s. 8d. ¼ in., 31s.

Materials and Labour.

(Contracts up to £5000. Inc. 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—

Foundations, 2s. 2d. per cu. ft. Columns, 2s. 5d. per cu. ft. Beams, 2s. 5d. per cu. ft. Floor slabs 4 in. thick, 6s. 10d. per sq. yd.; Do., 5 in., 8s. 7d.; Do., 6 in., 10s. 3d.; Do., 7 in., 12s. Walls 6 in. thick, 10s. 3d. per sq. yd. Add for hoisting 3s. 6d. per cu. yd. above ground floor level. Add for rapid-hardening Portland cement 2s. per cu. yd.

REINFORCEMENT.—Mild steel round bars, including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 48s. 6d. 7/16 in. to ½ in., 43s. 6d. ⅝ in. to 2½ in., 42s.

SHUTTERING AND SUPPORTS.—

Walls, 155s. per square.

Floors (average 10 ft. high), 160s. per square. In small quantities, 1s. 8d. per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 1s. 8d.; in narrow widths, 2s.

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2s.; in narrow widths, 2s. 2d.

Raking, cutting, and waste, 5d. per lin. ft.

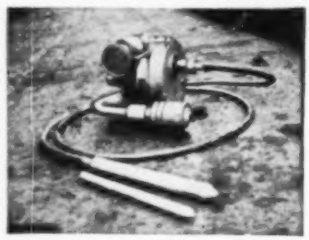
Labour on splays, 3d. per lin. ft.

Small fillets to form chamfers, 6d. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. per hour (carpenters 2d. a day tool money); Labourers, 2s. 6d.; Men on mixers and hoists, 2s. 7d.; Bar-benders, 2s. 8d.

This column is specially compiled for "Concrete and Constructional Engineering," and is strictly copyright.



Published by Concrete Publications Ltd., 14 Dartmouth Street, London S.W.1. Telephone: Whitehall 4581
Printed by Butler & Tanner Ltd., The Selwood Printing Works, Frome and London